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September, 1960

Papers

DISCUSSION

(over)

Journal of the

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Proceedings of the American Society of Civil Engineers

USE OF ALGAE IN REMOVING PHOSPHORUS FROM SEWAGE

By R. H. Bogan,[1] O. E. Albertson,[2] J. C. Pluntze[3]

SYNOPSIS

Eutrophication of receiving waters by nutrient rich wastes may be controlled by removal of phosphorus. Phosphorus can be removed from sewage by biological and chemical means. Either approach is aimed at converting soluble phosphorus to easily recovered insoluble particulate matter. An attempt was made during the period from 1957 to 1960 to exploit the metabolic activities of algae in removing phosphorus from treated sewage. A tertiary stage treatment process designed to strip phosphorus from solution through use of algal photosynthesis was developed in the laboratory. The process was subsequently studied in both laboratory and field scale pilot plants, and the basic principles of this process are outlined. Research leading to process development is briefly described, and process performance is evaluated and analyzed.

INTRODUCTION

Excessive enrichment or eutrophication of receiving waters by nutrient-rich wastes is emerging as a major water pollution problem. It has been recognized for some time that ordinary domestic sewage is a rich source of the nutrients

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required by phytoplankton. Experience has shown that the degree of eutrophication and hence the severity of subsequent water quality problems is largely dependent on the supply of inorganic nitrogen and phosphorus.

Attempts to control or prevent excessive plankton blooms have usually involved some manner of chemical treatment such as periodic application of $CuSO_4$, or diversion of nutrient-rich wastes to less sensitive or less valuable receiving waters, or a combination of these measures. Both of these control procedures have their limitations. Cost of the best available algicides precludes their use for continuous control of most eutrophic waters. Furthermore, the effect of most algicides is only temporary and does not get at the real cause of the problem. Diversion would appear to be a more effective and expedient course of action, but here, too, there are some serious shortcomings. As more and more drainage basins become involved, as now seems to be the case, the long range value of diversion grows increasingly doubtful. Moreover the economics of diversion are difficult to evaluate. They are frequently lost in the argument that there is no alternative solution.

From time to time interest has developed in some method of waste treatment which would remove offending fertilizing elements before discharge. Sawyer[4] has shown that removal of phosphorus offers a practical and effective way of controlling algal growths in most natural waters. Phosphorus removal may be accomplished by biological and by chemical means. Either approach is aimed at converting soluble inorganic phosphorus into recoverable insoluble matter. Of the two, chemical coagulation has received the greatest attention, and several promising but costly chemical treatment methods have been proposed.[5,6,7] Little has been done to date with biological treatment.

The concept of employing algae as a means of removing nutrients from domestic sewage is examined in this paper. Subsequent development of a treatment process employing a photosynthetic unit operation will be described. Laboratory and field scale pilot plant studies were made on removal of phosphorus from the effluent of an activated sludge plant.

THEORETICAL CONSIDERATIONS

Chemical Treatment Processes.—It is possible by means of chemical coagulation to reduce the soluble phosphorus content of sewage to concentrations as low as 0.10 to 0.50 mg per l. Lime, iron salts, filter alum and copper sulfate have been investigated.[4,5,6,7] Chemical requirements have been found to vary with the sewage being treated, with pH, and with the nature of the inorganic phosphate being removed. In general, optimum chemical doses have been found to range from about 150 mg per l to 400 mg per l or more.

The work of Lea, Rohlich, and Katz at the University of Wisconsin, Madison, Wis., indicates that alum and iron salts are equally effective in removing phosphorus. Optimum coagulant dose for secondary sewage treatment plant effluent was found to be in the vicinity of 200 mg per l. The effectiveness of vari-

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4 "Some New Aspects of Phosphates in Relation to Lake Fertilization," by C. N. Sawyer, Sewage and Industrial Wastes, 24, 708, 1953.

5 "Removal of Phosphates from Treated Sewage," by W. L. Lea, G. A. Rohlich, and W. J. Katz, Sewage and Industrial Wastes, 26, 261, 1954.

6 "Removal of Phosphorus from Sewage Plant Effluent with Lime," by R. Owen, Sewage and Industrial Wastes, 25, 548, 1953.

7 "Phosphates in Sewage and Sludge Treatment II Effect on Coagulation, Clarification and Sludge Volumne," by W. Rudolfs, Sewage Works Journal, 19, 178, 1947.

ous chemicals in reducing phosphate solubility is shown[5] in Table 1. Comparable phosphate reductions have been reported for lime doses ranging from 300 to 700 mg per l of $Ca(OH)_2$.[4,6]

The mechanisms of phosphate removal by chemical coagulation is not too well understood. Theoretically, phosphorus may be removed from solution through precipitation as an insoluble salt or by adsorption upon some insoluble solid phase. Available experimental evidence indicates that both mechanisms may be operative, particularly at low residual phosphorus concentrations. In the case of lime coagulation it appears that the principle mechanism is that of precipitation as insoluble calcium phosphate salts. With iron salts and alum, adsorption upon hydrated oxide floc-particles appears to play a major role. Pilot plant data indicate that floc settling properties may require much lower clarifier overflow rates than commonly employed in sewage treatment.

Cost appears to be the major limitation of all chemical coagulation processes studied to date. Based on available information the cost of chemicals alone would range from about \$25 to \$100 per million gallons of sewage treated for a 90% or more reduction in soluble phosphorus. Obviously the degree of phosphorus reduction will have some bearing on chemical requirements and attendant costs.

TABLE 1.—COMPARISON OF COAGULANT EFFICIENCY AT DOSES OF 200 mg PER l

Coagulant	Conc. Soluble P mg per l		Removal Soluble P, in percent
	Initial	Residual	
Alum	5.37	0.07	98.7
Ferrous Sulfate	5.67	0.06	99.0
Ferric Sulfate	6.02	0.05	99.0
Copper Sulfate	6.16	0.24	96.1

Biological Treatment Processes.—The concept of removing nutrients biologically can hardly be considered as new or unique. In any actively growing system nutrient materials are continually extracted from the environment through conversion to cell tissue. Rate of nutrient removal, other things being equal, is a function of the rate of cell tissue synthesis, whereas the amount of nutrient reduction is determined by cell tissue composition and the mineral content of the medium. Growth rates vary greatly with type of organism and with the species. The mixed microbial culture provided by the activated sludge process would appear to be the most effective biological system in terms of removal rate.

Examination of the data presented[8] in Table 2 indicates that assimilation of 1 mg per l of phosphorus by algae would be accompanied by metabolism of 33 mg per l to 78 mg per l of carbon and 1 mg per l to 12 mg per l, or more, of nitrogen. Similar considerations also apply to bacteria.

Ordinary domestic sewage does not provide a balanced diet. Both carbon and nitrogen are deficient with respect to the amount of phosphorus normally present. In the case of activated sludge, organic carbon is usually limiting. Adequate amounts of carbon are normally available to algae in the form of alkalinity. There also is evidence that atmospheric nitrogen fixation might

8 "Algal Culture from Laboratory to Pilot Plant," by R. W. Krauss, Carnegie Inst. of Washington, Publication 600, Chap. 8, Table 3, 1953, p. 90.

serve as a significant source of nitrogen in large scale algal cultures.[4] Viewed in terms of nutritional requirements algae appear to offer the most easily exploited biological system for extracting phosphorus from sewage.

Conventional biological sewage treatment should, of course, remove some phosphorus. With a BOD to phosphorus requirement in the range of 100 to 300: 1 for a typical aerobic reactor and a settled raw sewage BOD in the range of 100 mg per l to 200 mg per l it becomes obvious that phosphorus reductions during the course of complete biological treatment would, on the average, be limited to approximately 1 mg per l. Owen,[6] in an investigation of sewage treatment plant performance in Minnesota, found phosphorus removal ranged from an average 2% for primary treatment plants to an average 23% for plants employing biological treatment. This was equivalent to approximately 1 mg per l to 2 mg per l of phosphorus. Rudolfs[9] reported phosphorus reductions during the course of biological treatment running as high as 75% to 90%! Analysis of sewage treatment plants in the Seattle, Wash. area disclosed reductions ranging from 15% to 40%, which was equivalent to 0.80 mg per l to 2.0 mg per l of phosphorus.

TABLE 2.—COMPOSITION OF FRESH WATER ALGAE

Element	Per cent Total Dry Weight
Carbon	49.5 - 70.2
Oxygen	17.4 - 33.2
Hydrogen	6.6 - 10.2
Nitrogen	11.4 - 11.0
Phosphorus	0.9 - 1.5
Sulfur	.09
Magnesium	0.3 - 1.5
Calcium	0.0 - 1.5
Potassium	0.0 - 1.4

The Use of Algae in Sewage Treatment.—The general role of algae in sewage treatment has received considerable attention in recent years.[10,11,12] Unfortunately very little has been reported regarding nutrient reductions stemming from algal activity. Available information indicates phosphorus reductions ranging from 10% to 90% or more. Performance appears to be erratic and unpredictable. Considerable difficulty has been experienced in harvesting algal cell tissue. This difficulty, coupled with slow growth rate, would account for some of the wide fluctuations noted in the mineral composition of many oxidation pond effluents.

Recovery and Re-use of Algae.—A relatively simple process was envisioned whereby algae would be employed as a means of removing phosphorus from

9 "Phosphates in Sewage and Sludge Treatment I Quantities of Phosphates," by W. Rudolfs, Sewage Works Journal, 19, 43, 1947.

10 "Algal Symbosis in Oxidation Ponds—II Growth Characteristics of Chlorella pyrenoidosa Cultured in Sewage," by W. J. Oswald, et. al., Sewage and Industrial Wastes, 25, 25, 1953.

11 "Algal Symbiosis in Oxidation Ponds—III Photosynthetic Oxygenation," Ibid, 25, pp. 692-705, 1953.

12 University of Texas, Civil Engineering Department Development of Design Criteria for Waste Stabilization Ponds, Final Report to the AEC, March 1, 1957.

sewage. In essence, the process would consist of a growth cell or lagoon followed by a separation device for recovering algal cell tissue. Algae would be recycled for re-use or wasted to a sludge-holding lagoon according to need. Sewage would be mixed with actively growing algae in a lagoon or growth cell. The principal problem, at the outset, appeared to be one of recovering cell tissue.

A number of harvesting operations have been investigated. All have been found wanting in some aspect, generally in terms of cost, often in terms of efficiency. From the standpoint of complete performance and economy some type of screening device appears to be the most promising. Screening-performance would obviously be related to the nature of the algal culture. It was reasoned, however, that if some readily recovered alga could be established through the simple mechanism of recovery and re-use it could, in turn, be made to predominate. Thus, the process culture would be made to adjust to the most readily utilized population. Much of the early portion of this research was influenced by these concepts.

RESULTS

During the course of this investigation, a number of basic concepts came under study. A brief summary of some of the more significant findings are presented with emphasis on the questions of usable algal cultures, observed phosphorus removal, $PO_4^{\equiv}$ solubility, photosynthetic pH shift, and pilot plant performance.

Algal Cultures.—A search was made for usable algae with particular interest in large filamentous species. Cultures were grown in aerated 2 l glass tubes at several temperatures ranging from 10° C to 25° C. Light intensity was maintained at 400 to 500 ft-c. Various mixtures of lake water and raw or treated sewage were employed as seed and culture media. During most of the laboratory phases of this research in inorganic synthetic sewage was designed to approximate the composition of secondary sewage treatment plant effluents in the Seattle area; its composition is shown in Table 3.

Several common fresh water algae were grown. However, except for Chlorella and Scenedesmus, most types died after a brief period of growth. A large filamentous alga, subsequently identified as Stigleoclonium stagnatile, was recovered from the rock of a biological filter in the area and successfully cultured. This alga when grown under aeration developed into setteable floc particles resembling activated sludge. It subsequently became the subject of a large part of the laboratory phase of this investigation. Photomicrographs of Stigleoclonium stagnatile are shown in Fig. 1. Fig. 1(a) shows floc-like colonies which developed in aerated cultures. The magnification is 220 times. Fig. 1(b) shows a view of the individual organism. The magnification is 520 times. Its growth characteristics and nitrogen and phosphorus content are shown in Tables 4 and 5. By way of comparison, the growth rates of a number of other algae are shown in Table 5.

Observed Phosphorus Removals.—The rate at which an algal culture may be expected to extract phosphorus from solution should be a function of growth rate, cell tissue concentration, and the phosphorus content of the cell tissue. Computed theoretical metabolic uptake rates for cell tissue containing 2% phosphorus (dry weight) are shown in Fig. 2. Light intensity will determine cell tissue concentration. Temperature, diet and species will regulate k. In the

TABLE 3.—SYNTHETIC SEWAGE COMPOSITION

Substance (1)	Source (2)	Amount Added to Tap Water as Ion or Element, in mg per l (3)	Amount Present in Tap Water,[a] in mg per l (4)
N	$Ca(NO_3)_2$	10	—
	KNO_3	20	—
	or		
	NH_4Cl	30	—
P	KH_2PO_4	10	—
Na	$NaSiO_2$	5.0	1.91
	NaCl	39.0	
K	KNO_3	55.6	0.32
	KH_2PO_4	12.2	
Ca	$CaCl_2$	varies	7.11
	$Ca(NO_3)_2$	14.3	
Mg	$MgSO_4$	4.8	0.70
Fe	$FeSO_4$	0.20	0.05
Cl^-	$CaCl_2$	varies	1.15
	NaCl	30.0	
HCO_3^-	$NaHCO_3$	100.0	25.1
SO_4	$MgSO_4$	15.0	2.05
SiO_2	$NaSiO_2$	15.0[b]	8.0

[a] Based on analytical data obtained from city of Seattle Engineering Department for years 1951 - 1955.
[b] Reduced to 1.5 mg per l for pilot plant operation.

TABLE 4.—EFFECT OF TEMPERATURE AND CULTURE MEDIA ON GROWTH RATE STIGLEOCLONIUM STAGNATILE[a]

Temperature, °C (1)	Synthetic Sewage Employing NO_3-N (2)	Synthetic Sewage Employing NH_3-N (3)	Secondary STP Effluent (4)
	k $days^{-1}$[c]	k day^{-1}[c]	k $days^{-1}$[c]
10	0.165	0.140	0.170
15	0.188	0.179	0.215
20	0.252	0.131	0.131[b]

[a] pH varied from 8.3 - 9.5, illumination was at 400 ft c.
[b] _Scenedesmus_ appeared and began to predominate after approximately 15 days.
[c] k computed from $N_t = N_o e^{kt}$.

Seattle area algal concentration ranged from 25 to 50 mg per l in 4 ft deep lagoons. The mean k was 0.30 days.$^{-1}$ The soluble phosphorus content of most laboratory cultures was observed to decrease at a rate considerably in excess of that predicted by biological uptake alone. Response to repeated heavy doses of phosphate was most interesting. In general some 80% to 90% of the $PO_4^{\equiv}$ added

TABLE 5.—COMPARISON OF ALGAL GROWTH RATES[a]

Organism (1)	k^a days^{-1} (2)	Generation Time	
		days (3)	minutes (4)
Anabaena cylindrica	0.74	0.94	1350
Chlorella pryenoidosa	0.48 - 2.0	0.35 - 1.4	500 - 2070
Chlorella vulgaris	0.67 - 1.1	0.63 - 1.0	600 - 1500
Euglena gracilis var. bacillaris	0.79	0.71	1020
Scenedesmus quadricauda	2.0	0.35	500
Scenedesmus costulatus	0.28 - 1.08	0.64 - 1.4	920 - 2070

[a] $L_{ne} \frac{Nt}{No} = k\,t$

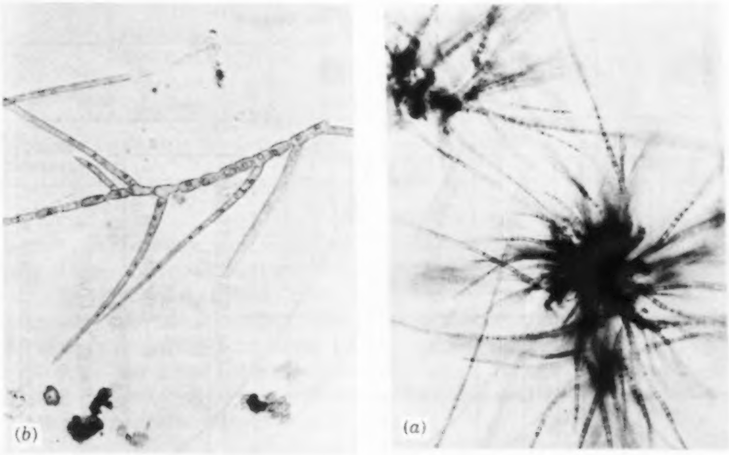

FIG. 1.—PHOTOMICROGRAPHS OF STIGLOECLONIUM STAGNATILE

was removed from solution within 2 hr as shown in Fig. 3. Aerated Stigleoclonium cultures exhibited a remarkable capacity for coagulating and adsorbing ortho phosphate. Photosynthetic response was not visably impaired by repeated use of cell tissue. Obviously more than metabolic uptake was involved.

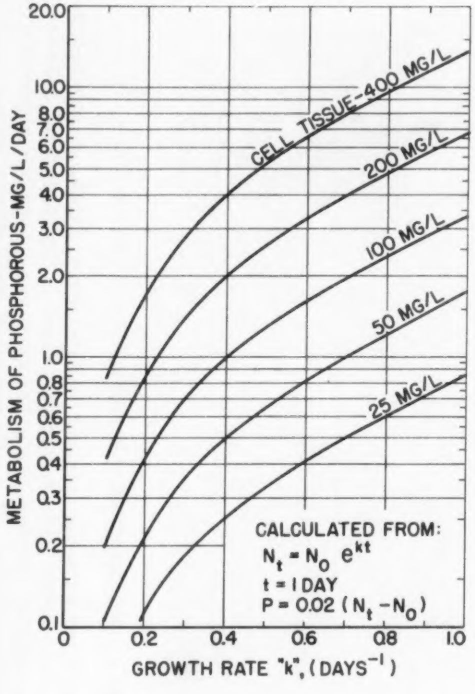

FIG. 2.—THEORETICAL RELATIONSHIP BETWEEN CELL TISSUE CONCENTRATION, GROWTH RATE AND METABOLIC CONVERSION OF PHOSPHORUS BY ALGAE

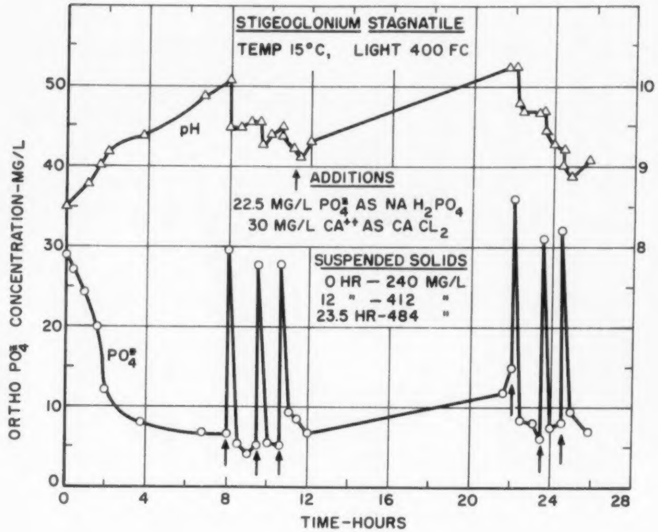

FIG. 3.—RESPONSE OF BATCH FED ALGAL CULTURES TO REPEATED DOSES OF $PO_4^{\equiv}$

Examination of culture characteristics disclosed that phosphorus removal was closely related to pH. It appeared that coagulation and adsorption may have played a major role. Accordingly, the next phase of the investigation was concerned with the physical chemical behavior of $PO_4^{\equiv}$ and particularly those factors affecting solubility.

Factors Affecting $PO_4^{\equiv}$ *Solubility.*—Orthophosphate may combine with a number of substances ordinarily present in sewage to form relatively insoluble complexes under suitable conditions. Calcium ion concentration and pH were found to be the principal controlling factors in determining $PO_4^{\equiv}$ solubility. The relationship between $PO_4^{\equiv}$ solubility, pH and Ca^{++} concentration is shown in Fig. 4. Calcium ion concentration and pH were adjusted by means of $CaCl_2$, $Ca(OH)_2$ and NaOH. Samples of synthetic sewage and sewage treatment plant effluent flocculated for 15 min. Phosphate residuals are based on filtered supernatatant. Ammonia, iron, and magnesium in amounts normally encountered in domestic sewage did not exercise any discernable effect on $PO_4^{\equiv}$ solubility.

Photosynthetic pH Shift.—A study was made of the role of algae in adjusting pH. This adjustment of pH is presumable related to consumption of CO_2 and an attendant shift in the carbonate-bicarbonate ion balance. Such factors as

TABLE 6.—PHOSPHOROUS AND NITROGEN CONTENT OF STIGLEOCLONIUM STAGNATILE HARVESTED FROM VARIOUS CULTURE MEDIA[a]

Constituent	Synthetic Sewage		Secondary STP	1/2 STP Eff.
	NO_3 - N	NH_3 - N	Effluent	1/2 NO_3 - N Syn Sew.
N	5.71	6.59	6.52	6.00
P	2.16	1.81	1.89	2.07
N/P	2.64	3.63	3.44	2.89

[a] Expressed as percent dry cell weight. Values reported represent an average of several determinations.

growth rate, cell tissue concentration, temperature, light intensity, and the alkalinity of the culture medium would be expected to influence rate of pH change. Interestingly, light intensity was found to be the principal rate controlling factor. Repeated experiments indicate that where mean culture light intensity was 100 to 200 ft-c and above variation in cell tissue concentrations from 50 mg per l to 400 mg per l had little effect on the rate of change of pH. Also, it was noted that increasing light intensity above 200 ft-c had little influence on the rate of pH change, other factors being equal.

The pH response of a Scenedesmus culture in the presence of increasing concentrations of HCO_3^- is shown in Fig. 5. At light intensities above 200 ft-c Scenedesmus, Chlorella and Stigleoclonium caused a rapid increase in the pH of raw and treated sewages. Alkalinity in amounts generally encountered in most sewages had little affect on pH response. These data are typical where light was not a limiting factor. Generally speaking, the pH level of secondary plant effluent would increase to 9.0-0.1 in 4 to 6 hr. Much higher pH levels were ultimately reached as these data indicate; however, there was usually a lag period of 6 to 12 hr preceeding the second increase in pH level.

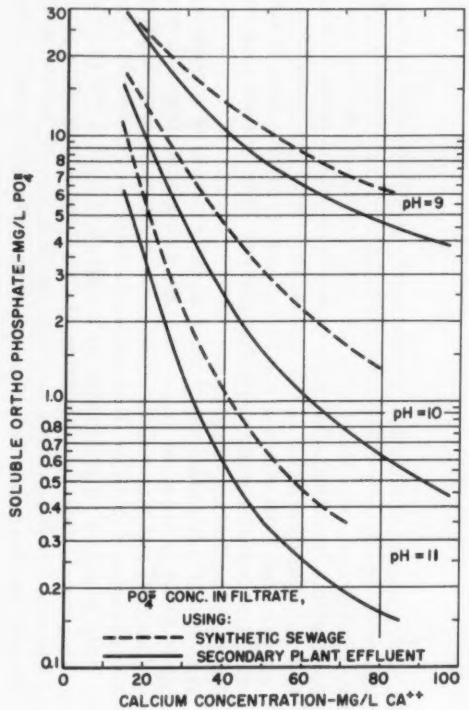

FIG. 4.—INFLUENCE OF pH AND CALCIUM CONCENTRATION ON ORTHOPHOSPHATE SOLUBILITY

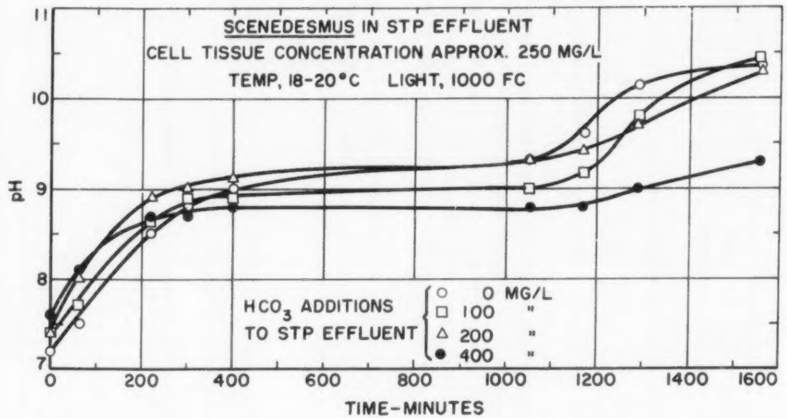

FIG. 5.—INFLUENCE OF HCO_3^- CONCENTRATION ON PHOTOSYNTHETIC pH SHIFT

Data presented in Table 7 describe photosynthetically induced pH changes observed in open basins during field pilot plant studies. Mean light intensities through the algal culture were on the order of 10 to 20 ft-c. Liquid depth was approximately 3 1/2 ft. These data are typical of raw sewage lagoons and oxidation ponds in the Seattle area during the summer and early fall.

Laboratory Pilot Plant Studies.—The forementioned findings were subsequently translated into a laboratory scale pilot plant. This pilot plant consisted of an illuminated contact unit, a clarifier, and a basin or lagoon for regeneration of algal cell tissue. A flow diagram of the laboratory pilot plant is shown in Fig. 6.

The process was operated on both synthetic sewage and on effluent from an activated sludge plant in the area. Contact time, recirculation ratio and the

TABLE 7.—PHOTOSYNTHETIC pH CHANGES IN LAGOON CELLS - AUTUMN 1958

Lagoon Cell	Time, in days								Remarks
	1	2	3	5	7	10	12	15	
1	7.7	7.6	8.4	8.7	8.5	9.4	9.5	10.1	Intermittent aeration plus artificial illumination (400 ft_e-c) PM
2	8.0	7.7	8.4	8.8	8.9	9.2	9.2	9.6	Int. aeration: no artificial illum.
3	8.1	8.0	8.6	9.1	9.3	9.7	9.7	10.4	No.aeration; no art. illumination
4	8.1	7.5	8.2	10.4	—	—	—	—	Int. aeration. Air off after day 6
Time of Sampling	1:30 pm	11 am	11 am	11:30 am	10:30 am	3 pm	11:30 am	12 am	All lagoon cells filled with secondary treatment plant effluent and seeded with equal volume of algal culture
Weather	Cloudy	Cloudy	Pt. Cldy	Rain	Sunny	Pt. Cldy	Sunny	Sunny	
Temperature °C	—	—	14	—	12	11	10	11	
Suspended Solids mg per l	—	28	33	36	36	62	62	70	

effect of lagoon or regeneration basin operation were studied. Typical performance data are shown in Fig. 7. Actual $PO_4^{\equiv}$ removals have been compared with theoretical values computed from the data presented in Fig. 4.

The significance of Ca^{++} concentration and of pH is obvious. It also was found that sludge recirculation ratios up to 3:1 were beneficial and that little was to be gained by increasing holdup time in the contact unit beyond 4 hours in the presence of 200 ft-c or more of light.

When the process was operated on secondary sewage treatment plant effluent, the Stigleoclonium culture became contaminated with Chlorella and Scenedesmus. At temperatures above 20° C, Chlorella and Scenedesmus tended to predominate; the Stigleoclonium was simply overgrown. This gave rise to some misgivings at first until it was observed that settling characteristics

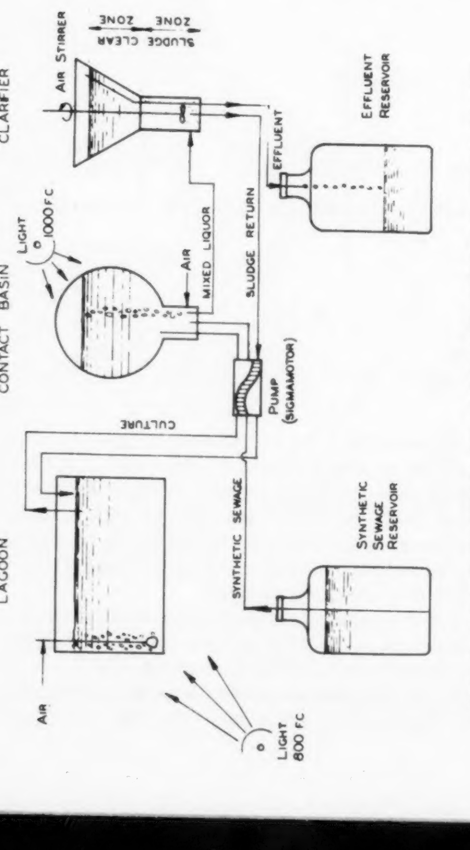

FIG. 6.—FLOW DIAGRAM OF LABORATORY SCALE PILOT PLANT

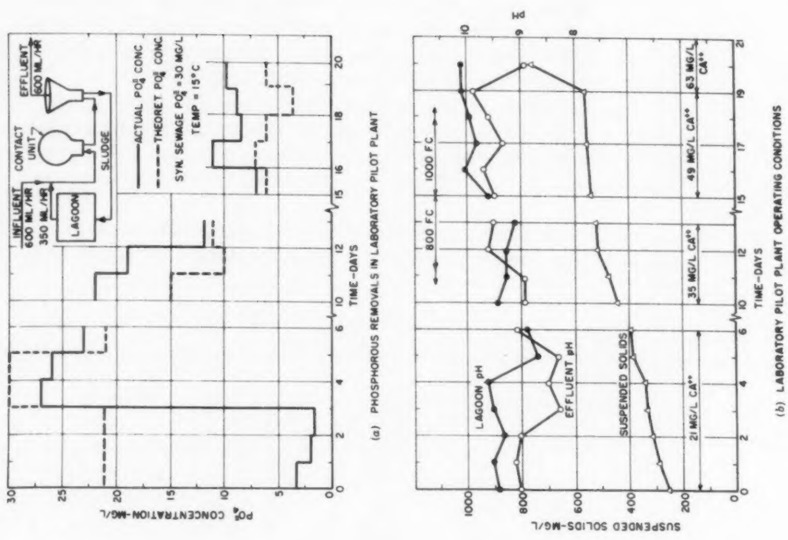

FIG. 7.—TYPICAL LABORATORY PILOT PLANT PERFORMANCE

varied moderately with algal species owing to the coagulation effect of the insoluble phosphate salts produced at high pH levels.

Field Pilot Plant Studies.—A field scale pilot plant was constructed employing the findings of the laboratory studies as a basis for design. The plant was operated on the effluent from an activated sludge sewage treatment plant. The contact unit and clarifier each provided a two hours retention time and an overflow rate of 360 g per sq ft day at design flow of 10 gpm. The lagoon phase of the operation was designed to provide five days retention of the algal culture sludge anticipated in the process. Artifical illumination of the contact tank and the lagoon cells was provided by means of industrial fluorescent fixtures. The overall dimensions of the pilot plant were 32 ft by 16 ft by 4 ft deep. A flow diagram of the pilot plant is shown in Fig. 8. Fig. 9 shows the plant. The influent pump is in the foreground, the illuminated contact tank at the left and the lagoon cells to the right.

Inability to achieve a mean light intensity above 100 ft-c markedly inhibited rate of change of pH. The effect of this was to render the contact unit inoperative. Where adequate detention time was available, such as in the lagoon cells, suitable pH levels were subsequently realized as shown in Table 7.

When it became apparent that photosynthesis alone would provide the desired degree of pH adjustment, attention was directed toward supplemental use of lime. The effect of various lime doses was first studied in the laboratory. Lime doses giving the desired pH change or $PO_4^{\equiv}$ reduction were then applied in the field. Performance employing both lime and algae taken from lagoon cells is shown in Fig. 10. At light intensities prevailing in the Seattle area during the fall, rapid photosynthetic pH adjustment was not possible. Phosphorus removal was due entirely to the influence of lime. Actual $PO_4^{\equiv}$ residuals have been compared with theoretical values computed from measured pH and Ca^{++} concentration. The discrepancy between theory and actual performance appears to be related to sedimentation tank performance. Performance was in good agreement with theory since the maximum difference noted was equivalent to approximately 10% of the influent $PO_4^{\equiv}$ content.

ANALYSIS

An attempt was made to exploit the metabolic activities of algae in removing nutrients from treated sewage. A tertiary stage treatment process designed to strip phosphorus from solution through use of algal photosynthesis was developed in the laboratory. The process subsequently was studied both on a laboratory and on a field-scale pilot plant basis. Based on these studies it appears that the removal of inorganic phosphorus from solution by algae is the result of both metabolic uptake and of physical-chemical adsorption and coagulation. Adsorption and coagulation appeared to play the major role where rapid removal of large concentrations of $PO_4^{\equiv}$ was involved. The relative significance of metabolic uptake naturally depends on environmental conditions and upon time available for growth. In either case, photosynthetic activity governed rate and extent of $PO_4^{\equiv}$ reduction.

Controlling Variables.—Both Ca^{++} concentration and pH combine to act as the principal regulator of phosphate solubility. Photosynthesis in turn serves to regulate pH. Thus any factor which serves to control algal growth rate will

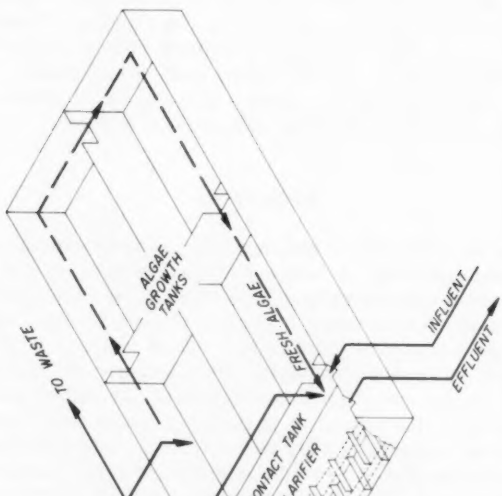

FIG. 8.—FLOW DIAGRAM OF FIELD PILOT PLANT

FIG. 9.—FIELD SCALE PILOT PLANT

also regulate process performance. Several potentially controlling factors immediately come to mind including temperature, algal species, chemical constitution of the sewage being treated, and light intensity. Of these, light intensity was found to be the principal controlling factor. Where light was not limiting, the process appeared to be functionally sound, and was affected but little by other variables.

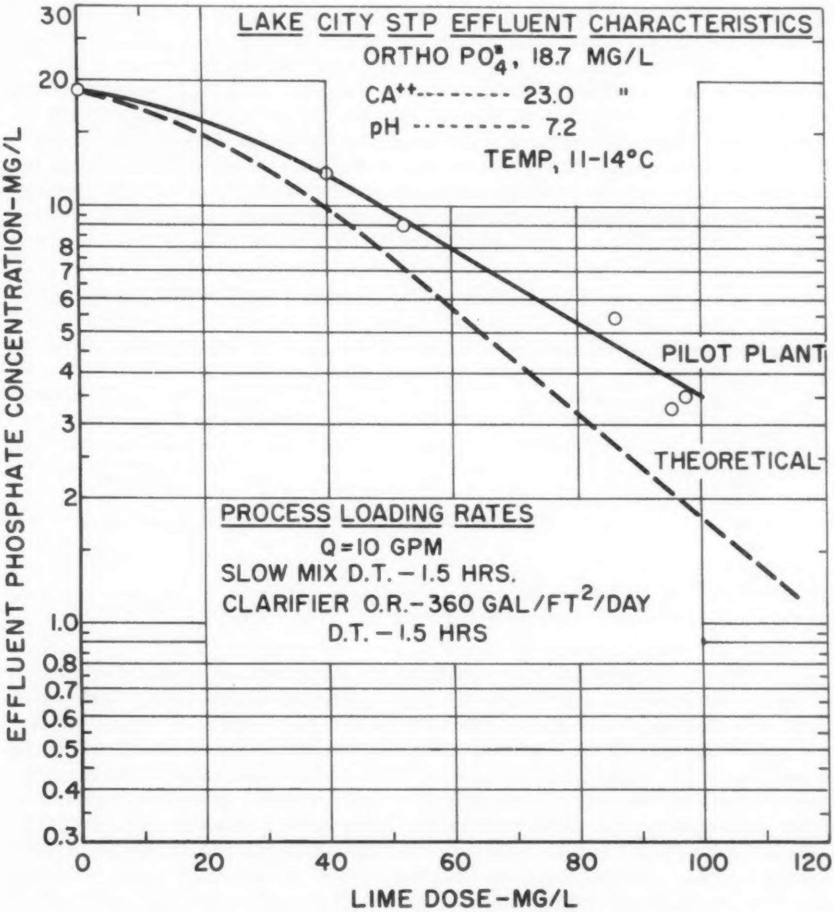

FIG. 10.—COMPARISON OF PILOT PLANT PERFORMANCE WITH THEORETICAL PHOSPHATE SOLUBILITIES

Light Requirements.—Most investigators have found optimum light intensities for algal cultures to lie in the range of 200 to 400 ft-c. Increasing this intensity beyond this value generally has little beneficial influence. Intensities above 1000 to 2000 ft-c often tend to inhibit photosynthesis. The results of this research suggest that adequate algal response as measured in terms of rate of change of pH was obtained at light intensities above 100 ft-c. It also

appears that rate of change of pH at light intensities above 100 ft-c varies little as algal cell tissue concentrations are varied from 30 mg per l to 400 mg per l.

Optimum light requirements are easily determined experimentally in the laboratory. Unfortunately the problem of maintaining such conditions on a large scale is more difficult. The severity of the lighting problem is determined entirely by the light transmitting properties of the algal culture. The light transmission or adsorption characteristics of an algal culture may be described by the Beer-Lambert Law.

$$I_t = I_o e^{-Ecd} \quad \dots\dots\dots\dots\dots\dots\dots (1)$$

in which I_o is the initial light intensity entering the culture, I_t is the intensity of the transmitted light at any depth d, E is the extinction coefficient in square centimeters per milligram, c is the concentration of algae in milligrams per liter, and d is the depth expressed in centimeters. Values of the extinction coefficient were found to range from 2.0×10^{-3} to 3.9×10^{-3} cm^2 per mg, which is in substantial agreement with values reported by Oswald[13] (1.0×10^{-3} to 2.0×10^{-3}) and that reported by Tamiya[14] (3.8×10^{-3}).

TABLE 8.—RELATION[a] BETWEEN Ig, ALGAE CONCENTRATION, AND DEPTH at I_t = 100 ft_6-c

Concentration Algal Cell Tissue mg per l	Depth - cm for Corresponding I_o			
	1000 ft_{-c}	2000 ft_{-c}	5000 ft_{-c}	10,000 ft_{-c}
(1)	(2)	(3)	(4)	(5)
50	23	30	39	46
100	11.5	15	19.5	23
200	5.8	7.5	9.8	11.5
400	2.9	3.8	4.9	5.8

[a] I_t computed from $I_t = I_o e^{-Ecd}$; $E = 2 \times 10^{-3} \frac{cm^2}{mg}$.

In order to demonstrate the significance of light attenuation, the distances at which various light intensities would penetrate different concentrations of algae and still leave a residual intensity of 100 ft-c have been computed, and are shown in Table 8. Sunlight may provide 1000 ft-c to more than 10,000 ft-c, depending on the time of day, season of year, and geographical location. Most commercial lighting units are capable of supplying light intensities of 2000 ft-c and less. Inspection of the data shown in Table 8 discloses that (a) relatively little is to be gained by markedly increasing incident lighting intensity, and (b) the photosynthetically active depth of most algal culture is generally less than 1 ft. When it is recalled that light intensities much in excess of 1000 ft-c to 2000 ft-c may actually be inhibitory, the futility of attempting to provide adequate illumination solely through increasing initial intensity, becomes all the more apparent.

Thus, in order to maintain maximum photosynthetic activity on a large scale it would be necessary to either restrict depths to a maximum value on the order

13 "Photosynthesis in Sewage Treatment," by W. J. Oswald and H. B. Gotaas, Proceedings—Separate No. 636, ASCE, May, 1955.

14 Carnegie Inst. of Washington Publication 600, Algal Culture from Laboratory to Pilot Plant, Washington D. C., 1953.

of one foot, or to have sources of illumination located throughout the culture mass spaced, in general, less than 2 ft apart.

Most lagoons and oxidation ponds are constructed to operate with liquid depths of 3 ft to 5 ft. Obviously, only a portion of the liquid depth may be considered as a zone of active photosynthesis. As a practical matter, where thorough mixing of pond volume takes place, the effect of employing depths greater than that involved in photosynthesis is roughly equivalent to illuminating the entire culture at a proportionally lesser intensity. The net photosynthetic response is thus reduced, and a longer contact time must be provided to achieve a given degree of pH adjustment. For example, it was found that 12 hr contact at 200 ft-c was approximately equivalent to 10 to 12 days holdup in a 3 1/2 ft deep lagoon under field conditions prevailing in the Seattle area during the fall.

Economic Evaluation.—When artificial illumination is employed, the cost of photosynthetic pH adjustment is determined by power requirements and by space or volume needs. A finite culture volume will be effectively illuminated by each light element and any portion of the culture lying outside this volume or at some greater distance will receive inadequate amounts of light. If the light intensity and power input of the light source together with the light transmitting properties of the culture are known, it is possible to express light requirements in terms of power per unit culture volume. This in turn may be translated into actual lighting cost if the illumination period is known. The cost of illuminating cultures by means of commercially available fluorescent elements has been computed and is shown in Fig. 11. Cost of electrical energy was taken at \$.01 per kw-hr. Power requirements were based on high voltage fluorescent elements and a culture extinction coefficient of 2×10^{-3} cm^2 per mg. Oxidation pond costs were based on a construction cost of \$0.05 per cu ft, a usefull life of 20 yr, and a liquid depth of 4 ft. Lime requirements were based on titration curves for sewages in the Seattle area. Mechanical equipment costs were neglected.

The cost of employing sunlight is determined by the cost of providing the necessary detention time. Detention times ranging from 2 to 10 or more days may be required to reach pH levels of 9.5 and above. It is contemplated that construction similar to that employed in oxidation ponds and lagoons would be used in this case. The cost of photosynthetically adjusting pH by means of sunlight, has been computed (Fig. 11) based on employing lagoons having an average depth of 4 ft, a unit volume cost of \$0.05 per cu ft, an expected life of 20 yr, and response equal to that observed in these studies (Table 7).

The cost of adjusting pH by means of $Ca(OH)_2$ is also shown in Fig. 11. Lime requirements were computed from titration curves obtained for sewage treatment plant effluents throughout the Seattle area. The cost of lime was taken at \$0.02 per lb. Inspection of these cost data suggest that the use of presently available means of artifical illumination is not economically feasible. Furthermore, it would appear that even under the most favorable of circumstances, natural illumination would be no more economical than lime. Of course, where adequate volume already exists in the form of oxidation ponds or sewage lagoons the question then becomes one of determining the cost of adapting and employing what is already available.

As mentioned earlier, the field scale pilot plant was operated employing both algae and lime. The cost of removing different amounts of $PO_4^{\equiv}$ employing algae only and algae in conjunction with various lime doses is shown in Fig. 12. Costs shown are for power and chemicals only. These costs are bases on employing artifical illumination together with sunlight to achieve photosynthetic pH adjustment.

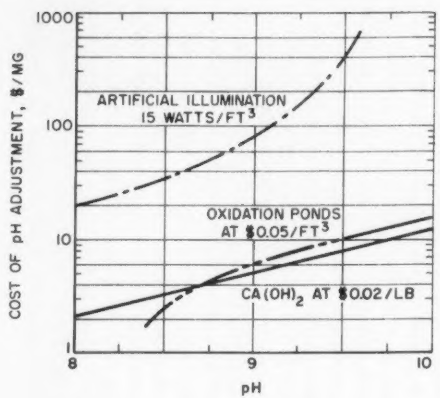

FIG. 11.—ECONOMIC COMPARISON OF VARIOUS METHODS OF ADJUSTING pH.

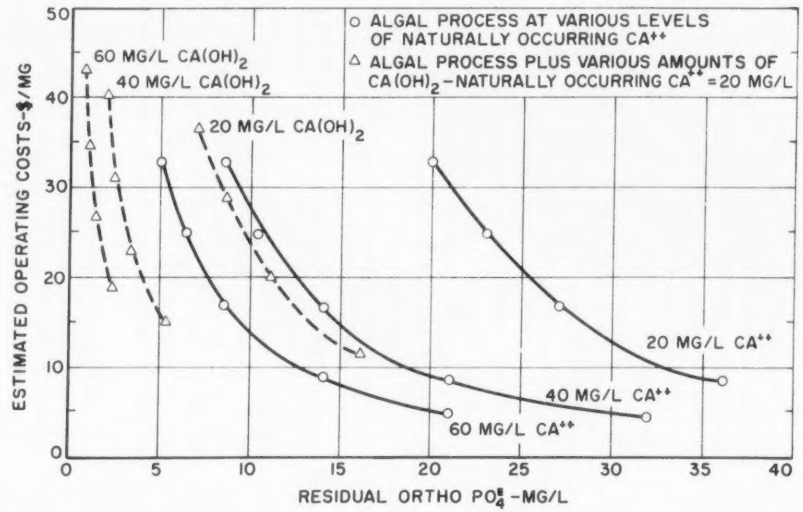

FIG. 12.—COMPARISON OF TREATMENT COSTS EMPLOYING HIGH RATE PROCESS

It should be noted that the concentration of naturally occurring calcium has a marked influence on treatment cost. This is readily understood when one recalls the relationship between pH, Ca^{++} concentration, and $PO_4^{\equiv}$ solubility shown in Fig. 4. Obviously, the mineral content of a community's carriage water may significantly alter the cost of biologically removing $PO_4^{\equiv}$, other things being equal.

The Use of Oxidation Ponds.—Theoretically, an oxidation pond should be capable of very high efficiencies of phosphorus removal. In order to realize this potential active photosynthesis and/or high pH conditions must prevail for some time prior to discharge. Where biological fixation is the principal mode of phosphorus removal, efficiency will be a function of detention time, growth rate, and cell tissue concentration.

During field pilot plant studies algal cell tissue concentrations in lagoon units having a depth of 3 ft to 4 ft varied from 25 mg per l to 50 mg per l. Average growth rate was equivalent to a k of 0.30 day^{-1}. Thus, in order to biologically extract 5 mg per l of P (equivalent to 80% to 90% reduction for most sewages) it appears that lagoon retention times on the order of 14 days to 28 days would be required. Theoretical retention time requirements for any other set of circumstances can be computed from the data presented in Fig. 2. These considerations suggest that detention times in excess of those commonly employed in most oxidation ponds may be necessary where a high degree of phosphorus removal is a treatment objective.

SUMMARY AND CONCLUSIONS

The possibility of employing algae as a means of biologically removing $PO_4^{\equiv}$ from domestic sewage was studied in the laboratory, and in the field, on a pilot plant scale. Early phases of this work were aimed at developing a process intended for use as a tertiary treatment stage wherein algae served as the sole means of $PO_4^{\equiv}$ removal. Under carefully controlled laboratory conditions, such a process was found to be functionally sound. Inability to maintain adequate light intensities under field scale conditions gave rise to relatively high treatment costs. The process as originally conceived was subsequently modified so as to include both the use of lime and the effect of natural illumination. The more significant aspects of this research are summarized as follows.

1. In the presence of adequate amounts of light it is possible to realize rapid biological extraction of $PO_4^{\equiv}$. For example, in a laboratory pilot plant orthophosphate concentration was reduced from 20 or more mg per l to less than 5 mg per l in less than 4 hr.
2. Adsorption and coagulation appear to play the major role where rapid removal of large amounts of phosphate is involved. Metabolic conversion is the principal mechanism of removal under the more leisurely conditions prevailing in oxidation ponds and sewage lagoons.
3. Three algae, Chlorella, Scenedesmus, and Stigleoclonium stagnatile, were grown in raw and treated sewages. Each was capable of rapidly increasing culture pH when illuminated at light intensities above 100 ft-c. Stigleoclonium grew in floc like particles resembling activated sludge.
4. Repeated use of cell tissue markedly improved subsidence properties without noticably impairing photosynthetic response. Settleability appeared to be influenced by the formation of insoluble $PO_4^{\equiv}$ compounds at elevated pH. Thus it appears that in addition to the prospects of removing $PO_4^{\equiv}$ there is also

the possibility that the photosynthetic pH shift may be employed as a means of enhancing the recovering of algae by sedimentation.

5. Light intensity was found to be the principal controlling variable. Under normal field conditions adequate light intensities seldom prevail in an algal culture at depths greater than 1 foot. The use of deeper ponds, as is common practice today, serves in effect to decrease the net mean illumination in proportion to the ratio of light to dark volumes.

ACKNOWLEDGEMENTS

The initial portions of this research including the laboratory pilot plant studies were supported by the University of Washington Engineering Experiment Station. The city of Seattle provided funds for construction of the field scale pilot plant.

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SANITARY ENGINEERING DIVISION
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LOW PRESSURE AERATION OF WATER AND SEWAGE

By N. Claes H. Fischerström,[1] F. ASCE

SYNOPSIS

Based on simplified theoretical considerations, a calculable system utilizing dispersed air at shallow depths has been developed. Elaborate tests and full scale operation have proved that it is possible to obtain a good economy, and an extremely high oxygenation capacity with this principle and perforated pipes as air distributors. Clogging of the pipes has been studied and satisfactorily eliminated.

INTRODUCTION

Oxygenation of a liquid, such as water, requires contact between air (or pure oxygen) and the water. This can be achieved by creating dispersion of the air into the water, or by spreading drops of the water into the air directly or by "coating" the water around particles, or by combining these methods. There are a number of devices adapted to create such contact, usually divided into the compressed air types, and the mechanical surface or similar aerators. Usually distribution of finely divided air into water is called air diffusion, but this term ought to be avoided because diffusion is a process occurring within one phase. All types introducing air under pressure (or suction) such as aerators with porous or perforated distributors, turbines, injectors, impingement bowls, Venturi-diffusors, and so on, are included here in the first group and then denoted as "dispersed air" types. Among dispersed air aerators it is

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Note.—Discussion open until February 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. SA 5, September, 1960.

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possible to separate between fine, medium size, and coarse bubble aerators. These categories give the best idea of the character of the aerator.

At present the dispersed air system with compressed air and porous tube or plate distributors as "diffusers" is most widely adopted for aerating of sewage. Generally, the aeration takes place in long basins with square or rectangular cross section, and the distributors are arranged longitudinally at, or near, the bottom. In the United States the distributors are usually located at one side of the basin. When sewage is flowing through the basin and compressed air is introduced through the porous distributors, a "spiral-flow" is supposed to set in from the inlet to the outlet end. Another type, generally used in England, employs the dispersed air system, with porous plates in rows spaced over the bottom area of the basin, as at the "ridge and furrow" bottom.

The porous distributors have been of fundamental significance for the development to its present status of the activated sludge process,[2] which was originally invented by E. Ardern and W. T. Lockett in 1913. Although they are difficult to clean of the porous materials, they seem to have been superior to most other "blowing-in" devices in this respect. At the present stage of development of the activated sludge process, the limited capacity, or perhaps the high investment cost of a large plate area at great volumes of air introduced with good economy, is of more importance.

The mechanical aerators do not usually involve clogging problems. However, even in the improved modern forms, they have a limited absolute capacity, and hence relatively low specific oxygenation rates when used as aerators for large, deep basins. Only the compressed air type aerators can give an unlimited total oxygenation capacity.

There is consequently a need for further investigation of the compressed air aeration technique, in order to find an economic high capacity aerator, without the disadvantages of the porous media aerators.

AERATION AT SHALLOW DEPTH

Borderline-Cases Considered.—It is generally accepted that when employing air dispersion it is necessary to introduce the air at, or at least near, the bottom, to get good power economy. E. P. Coombs[3] has stated: "Experiments by the late Mr. W. H. Makepeace and in America have proved that even for spiral flow tanks the diffusers should be on the floor for most efficient operation." The Sub-Committee on Air Diffusion[4] emphasizes the importance of getting a long contact period between the air bubbles and the water. The velocity of circulation must therefore be "kept near the practical minimum required to prevent sedimentation." Introducing air at a higher elevation than necessary above the bottom of a tank should be avoided because it reduces the "bubble contact area."

Aeration of water is an extremely complicated problem. There are many influencing factors that make it very difficult to draw general conclusions.

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[2] "Experiments in Oxidation of Sewage Without the Aid of Filters," by E. Ardern and W. T. Lockett, Jour. Soc. Chem. Ind., Vol. 33, No. 10, May 30, 1914.

[3] "Air Diffusers, Their History and Use in the Activated Sludge Process," by E. P. Coombs, Contr. Rec. & Publ. W. Eng., March, 1956.

[4] "Air Diffusion in Sewage Works," Manual of Practice No. 5, FSIWA, 1952.

The fundamental equation of oxygen absorption is

$$\frac{dC_t}{dt} = K\,A\,(C_i - C_t) \qquad (1)$$

in which K is assumed to be constant, and from which the equation which expresses the amount of oxygen absorbed by bubble dispersion may be written as

$$O = \int_0^t K\,A\,(C_i - C_t)\,dt \qquad (2)$$

In which O is the oxygen absorbed; A refers to the total contact area of bubbles; C_i denotes equilibrium concentration (saturation) of oxygen in the liquid at the gas-liquid interface; C_t denotes the concentration of oxygen in the liquid at the time t; and K is the coefficient of total oxygen absorption.

Eq. 2 looks simple, but difficulties arise when the influence of different factors such as temperature, depth of aerator, turbulence, quality of liquid, distribution of air, volumetric efficiency, and so on, is to be determined. An exact mathematical expression is not obtainable and even the empirical equations are still uncertain. A general treatment of aeration, though limited to shallow depth, would be extremely complicated. Some simplified, "calculable" cases with aeration at shallow depth will be studied in order to find the best oxygen uptake at a given applied effect.

At unchanged power consumption it is, of course, possible to use more air at a reduced aerator depth than at a greater depth. In a system with no friction losses the rate of air at the same input of energy is inversely proportional to the depth. Thus we can use from 7 to 10 times as much air at 0.5 m depth as at a normal depth. It is known that the absorption rate is very great at formation of the bubbles; however during a fraction of the initial second it decreases considerably to a low but fairly constant value, (Fig. 1).[5] Should we then test aeration by producing new air-water surfaces with large amounts of air at shallow depth? When varying the depth of an aerator with corresponding change of the amount of air, some important factors must be taken into consideration, especially the change in pressure, contact time, and the change in turbulence. It is known that the total oxygen absorption will increase almost linearly, at constant input of air with the depth of the aerator. Then the turbulence can be expected to be fairly constant. If the air rate is varied considerably, the turbulence will also vary.

The turbulence increase is very great within certain limits. Generally the turbulence increases in proportion to the amount of air. In this case it would increase inversely to the depth of aerator. The resulting effect of the two opposite influencing factors, depth and turbulence, will be an increase with decreasing depth. Only actual tests will show the influence of depth and turbulence.

In a system based on oxygenation through the production of new air-water contact surfaces, a high velocity of circulation might be beneficial, because at a high consumption rate the rearation must be repeated rapidly, and the increased turbulence created by the air not only divides the bubbles and the sludge but also improves the oxygen mixing and diffusion within the water and sludge.

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[5] "Research on Activated Sludge," by A. Pasveer, JSIW, November, 1953, p. 1253.

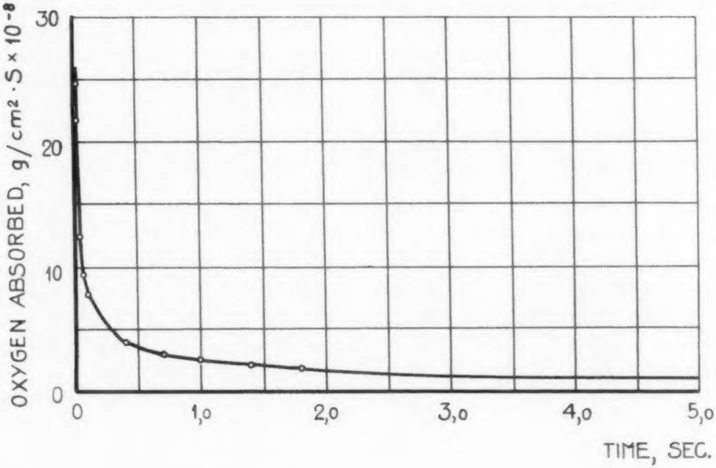

FIG. 1.—ABSORPTION RATE

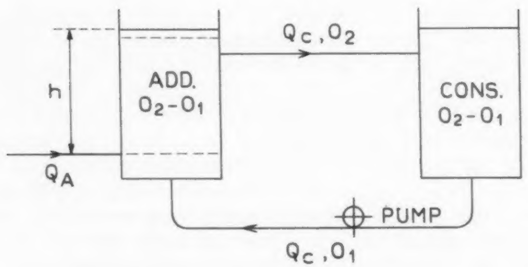

FIG. 2.—CIRCUIT OF CIRCULATING WATER

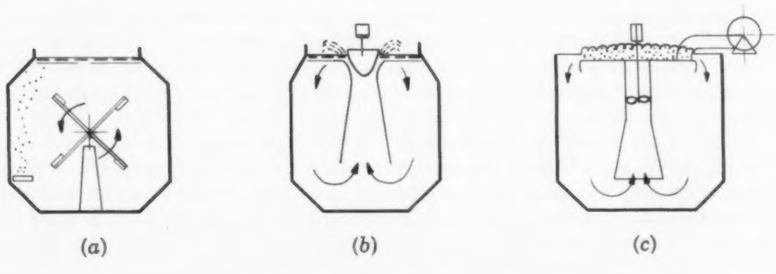

FIG. 3.—EXAMPLES OF AERATORS

At shallow depth and large amount of air, the loss of head in the aeration system will be of great importance. A. T. Ippen and C. E. Carver concluded after their studies[6] on oxygen transfer:

> "Absorption efficiency of oxygen is increasing with decreasing depth, while the influence of the diffuser loss becomes relatively greater. Overall efficiency nevertheless increases with depth until diffuser losses can be materially reduced."

Fig. 2 shows circuit of circulating water with one box in which oxygen is supplied and the other in which oxygen is consumed. Assume that the process is in equilibrium and that the oxygen content at the outlet of the consumption box is O_1, and at the outlet of the supply box is O_2 ppm. The circulation rate is assumed constant and independent of the aeration. An example of this principle is aeration combined with mechanical stirring[7] as in Essen-Rellinghausen. In the United States this is called Dorrco-Aerator (Fig. 3a). Other examples are Simplex (Fig. 3b) and Inka (Fig. 3c) aerators. We assume further that O_1 is very near zero and O_2 is below saturation, which is possible if the circulation is sufficient. All factors not to be studied are regarded as constant.

If the amount of circulating water is Q_C, the air supply is Q_A, the oxygen supplied, or, which is the same, the oxygen consumed, O_t, during the time t will be:-

$$O_t = Q_C \left(O_2 - O_1\right) t \quad \ldots\ldots (3)$$

in which

$$Q_C \left(O_2 - O_1\right) = k\, Q_A \quad \ldots\ldots (4)$$

If $Q_A = q_A$ at an aerator depth h = 1, the amount of air at constant input of effect will be

$$Q_A = \frac{q_A}{h} \quad \ldots\ldots (5)$$

Hence from Eqs. 3, 4, and 5

$$O_t = K \frac{t\, q_A}{h} \quad \ldots\ldots (6)$$

It is necessary to find the depth that gives maximum oxygenation. There is no real maximum, as seen from the equation; however asympototically, the oxygen supplied increases until h = O. This means that the amount of air Q_A increases to an infinite size. The oxygenation reaches a maximum degree (saturation), and the total oxygenation depends upon the circulation rate.

If the power needed for the circulation is neglected, dispersed aeration in an extremely shallow aeration, with large amounts of air, as in the "bubblelayer-aerator,"[8] is seen to be efficient and economical. The "bubblelayer-aerator" was an invention by J. O. Naucler, Stockholm. It consists[8] of a thin layer of water flowing over a perforated plate, through which is introduced relatively large amounts of air, so that all the water is transformed to bubbles, with an

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[6] "Basic Factors of Oxygen Transfer in Aeration Systems," by A. T. Ippen and C. E. Carver, JSIWA, July, 1954, p. 826.

[7] "Sewage Treatment," by K. Imhoff and G. M. Fair, New York, 1940, p. 156, Fig. 53.

[8] "Intensive Aeration of Water," by N. K. G. Westberg, JAWWA, Vol. 41, No. 5, May, 1949.

enormous air-water surface. This aerator has been used in activated-sludge plants with success, but it has the disadvantage of spreading drops of the sewage relatively high into the air, and hence creates risk of infection. Special mechanical equipment is also needed for bringing about the circulation. A bubblelayer of such an aerator is shown in Fig. 4. (The photo was taken in 1/10,000 of a second.)

In practice we cannot neglect the power required for the circulation of the water. If the circulation as is customary with "diffused air," should be produced by the aeration alone, there would be a decreased effect at very shallow depths of the aerator. The pumping effect would be small and the circulation losses comparatively great. When the air distribution system is located at shallow depth, the circulation itself will be the dominant factor.

Aeration at Shallow Depth by Application of Selfcirculation.—It is easily understood that if the circulation itself plays a role, the air distributor must cover the whole section of the rising water flow in order to obtain the highest

FIG. 4.—BUBBLE-LAYER OF AN AERATOR

efficiency. Oxygen cannot be introduced in water to an arbitrary concentration, because it is limited by the saturation point and an over-aeration of a narrow section only results in loss of air. It is assumed that the aerator is distrubuted over about half the width of the aerating channel, and that a longitudinal wall fprces the circulating water mainly downwards in the one section and mainly upwards in the other, Fig. 5(a).

Absorption of oxygen is often assumed to occur in three different phases. These are (1) at the formation and growth of the bubbles, (2) at the rising of the bubbles, and (3) at the bursting of the bubbles at the water level. The consumption of oxygen by biological action occurs at constant rate and one "cycle" of the oxygenation might be illustrated by Fig. 6.

Absorption can take place only below the saturation point. Maximum absorption is obtained when the water enters and leaves the distribution system with the lowest possible oxygen content. It is assumed that the circulation and the

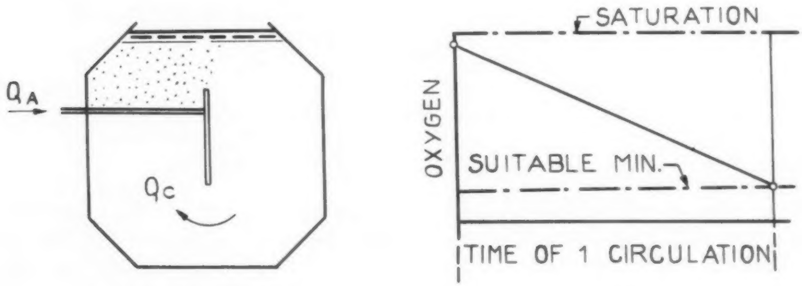
load and micro-organism activity is such as to satisfy these conditions, and that the system is in continuous equilibrium.

The computations regarding the oxygen absorption car carried out under very simplified assumptions. Their purpose is only to give general indications, regarding the rational design of an aeration system based on air dispersion.

Figs. 5(a) and 5(b) show an idealized spiral flow air-dispersion system and a very simplified picture of the oxygen cycle at every circulation of the water.

New water-air surfaces are directly proportional to the amount of air Q_A at a given size of the air bubbles. At a fixed power input, it is possible to only increase the amount of air through decreasing the depth of the distributor. The circulation of water, Q_C, will depend upon the amount of air, depth of distributor, and other factors.

The oxygenation is, however, not merely proportional to the amount of air Q_A and the rate of circulation Q_C. H. R. King[9] and Ippen and Carver[6] have verified that the oxygen absorption at constant air supply (that is, by influence of depth, time, burst, and so on) is more or less directly proportional to the depth, h. Increased amounts of air will exert additional influence by the in-



(a) (b)

FIG. 5.—IDEALIZED SPIRAL FLOW AIR-DISPERSION SYSTEM

creased turbulence. The turbulence increases with the amount of air and increase the "bubble-formation absorption coefficient." On the other hand, Ippen and Carver have shown that an increased rate of air at a given nozzle decreases the total absorption coefficient. It is assumed here, however, that the flow per nozzle is constant: that is, that the number of air openings is kept proportional to the amount of air, and thus eliminating the decreasing effect. The influence of air-rate and such factors as depth, time, etc., are assumed to be proportional to Q_A and h, respectively. The influence of turbulence might be a function of Q_A. For simplicity direct proportionality to Q_A is assumed and the oxygenation will probably be:

$$O_1 = \left(k' Q_A Q_C t\right) \left(k'' h\right) \left(k''' Q_A\right) \quad \text{. . . . . (7a)}$$

[9] "Mechanics of Oxygen Absorption in Spiral Flow Aeration Tanks," by H. R. King, I-III, JSIW, August, September, October, 1955.

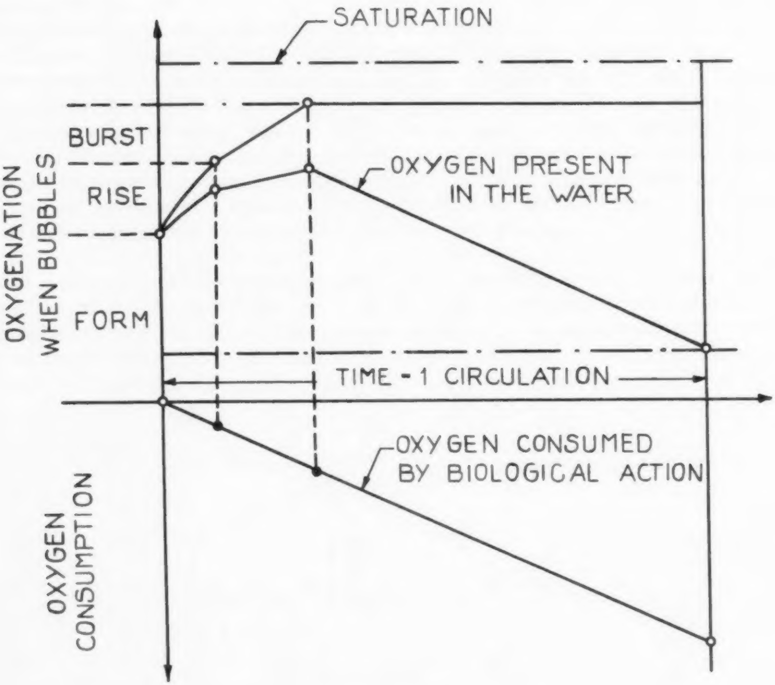
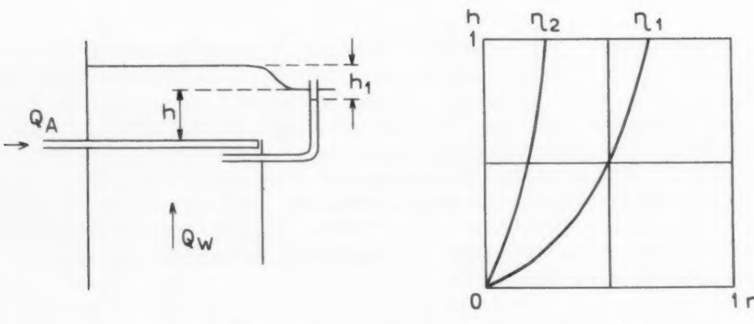


FIG. 6.—OXYGENATION CYCLE



(a) (b)

FIG. 7

or

$$O_1 = k_1 \, Q_A{}^2 \, Q_C \, h \, t \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(7b)}$$

in which O_1 is the oxygenation per unit of length and time t in seconds; Q_A refers to the amount of air per second and unit of length; Q_C denotes the amount of circulating water passing the aerators per second and unit of length; and k_1 is a constant. The author is aware that this equation can be criticized, but it should only be regarded as a guide in carrying out tests to confirm or to disprove the hypothesis made.

As previously pointed out, in a friction-free, air-distribution system, the amount of air is inversely proportional to the depth of the distribution system at constant input of energy and blower efficiency as indicated by Eq. 5. Hence the equation:

$$O_1 = k_1 \frac{q_A}{h} Q_C \, t \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(8)}$$

Within certain limits the water circulation, Q_C, is dependent on the amount of air and might be computed by the formula for the mammoth-pump, (Fig. 7). This formula is

$$Q_C = \frac{Q_A}{h_1}\left[\eta \, h - (1 - \eta) \, h_1\right] \quad \ldots\ldots\ldots\ldots\ldots \text{(9)}$$

in which η is pump efficiency, and h_1 refers to pumping head. For $h > 2h_1$

$$Q_C = \frac{q_A}{h \, h_1}\left[\eta \, h - (1 - \eta) \, h_1\right] \quad \ldots\ldots\ldots\ldots\ldots \text{(10)}$$

The over-all efficiency, η, is dependent upon the depth of the aeration system, the type of aerator, waterway, and so on. It is, of course, difficult to give reliable values of η (compare Fig. 7b). A maximum efficiency at 1 m depth of 60% to 70% and a corresponding curve at lower depth, as indicated by curve A and represented by $\eta_1 = 2 \, h \, (2h + 1)$, is assumed at low values of h_1. At nonfavorable conditions the efficiency might fall to 25% at 1 m and a correspondent curve at lower depth, indicated by curve B and $\eta_2 = h\!: (h + 3)$.

The oxygenation equation is then obtained.

$$O_1 = k_1 \frac{q_A{}^3}{h^2 \, h_1} \, t\left[\eta \, h - (1 - \eta) \, h_1\right] \quad \ldots\ldots\ldots\ldots \text{(11)}$$

The influence of the depth is found by derivation. For the limiting values of the first derivative we find (t = 1),

$$(\eta = \eta_1) \quad \frac{dO_1}{dh} = 2 \, K \frac{q_A{}^2}{h_1} \; \frac{h\left(h_1 + 3 \, h \, h_1 - 2h^3\right)}{h^4 \, (2 \, h + 1)^2} \quad \ldots\ldots \text{(12)}$$

$$(\eta = \eta_2) \quad \frac{dO_1}{dh} = K \; \frac{q_A{}^2}{h_1} \; \frac{h\left(18 \, h_1 + 9 \, h \, h_1 - h^3\right)}{h^4 \, (h + 3)^2} \quad \ldots\ldots \text{(13)}$$

The roots of these equations when $h_1 = 0.05$, are

$\eta = \eta_1$	$h_1 = O$	$h_2 = 0.38$	h_3 and h_4 imaginary
$\eta = \eta_2$	$h_1 = O$	$h_2 = 1.1$	h_3 and h_4 imaginary

(Actual tests had indicated that normally h_1 is 0.05 - 0.10 m, the size depending upon the amount of air and the shape of the waterway.)

The second derivative is > 0 at $h = 0$ and < 0 at h_2 - 0.36 and 1.1, respectively, indicating, as expected that under the assumptions made there is a minimum value when the aerator depth is zero and a maximum value at a depth somewhere between 0.4 and 1.1 m.

For different values of h_1 and, for $\eta = \eta_1$,

h_1	0.05	0.10	0.20
h	0.38	0.50	0.67
$h:h_1$	7.6	5.0	3.3

The most efficient depth is also influenced, to a large extent, by such factors, which determine the pumping effect.

Analysis.—It is known from earlier investigations[5] that if the air rate is kept constant the oxygenation increases proportionally with the depth of the distributor, as Fig. 8(a) indicates. The power will also increase as a straight line and the rate between oxygenation and power will be constant.

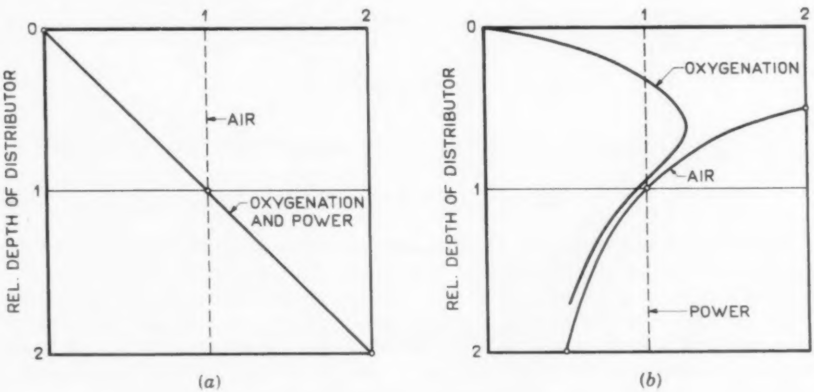


FIG. 8.—RELATIVE AIR POWER AND OXYGENATION

The computations would indicate that if the power is kept constant and the air rate is increasing inversely as the depth, the oxygenation will increase with decreasing depth to a certain point from which it will decrease to zero at zero depth, dependent upon the insufficient circulation. Fig. 8(b) shows curves for the oxygenation as described. Their shape depends upon several factors, such as air distribution, loss of head, influence of turbulence, and efficiency of the mammoth-pump circulation.

A conclusion, then, would be that at constant power it can be advantageous to use twice as much air at, say, 3 ft depth as at 6 ft depth, because of the increased oxygenation at the higher turbulence with 100% more air.

The computations indicate consequently that if the loss in the aeration system is negligible the economical depth of an aeration system could possibly be

small. This conclusion is drawn from computations based on approximations and simplifications, and, of course, only actual tests can show if the above is true in operation with pure water and with sewage, respectively.

Design Principles.—There seems to be three effects of special importance in obtaining an efficient low head aerator. These are (1) that caused in a quite definite zone of the depth of the distributor; (2) that caused by a partition wall; and, (3) that caused by an air distributor covering the whole cross section of the circulating rising water. In order to avoid longitudinal short circuiting due to tapered aeration or uneven air distribution at different parts of the aeration system the flow through the aeration basin must be sufficiently controlled.

With a system of this type above, it seems further to be important to get very low losses of head in the aeration system, and very low resistance in the water circulation system. A design meeting these requirements is shown in Fig. 9.

The aeration system in the upper part of the basin is spaced over a great width of the basin and rests on the partition wall with ample free openings at top and bottom, and is supported at suitable intervals by cross walls from the aeration system to the bottom of the basin. The aerator may consist of a grid of perforated pipes or of porous tubes or similar material. Very low loss of head and consequently a high amount of air can be obtained by the use of perforated pipes.

Some of the more important factors influencing the air "diffusion" in water have been investigated by Ippen and Carver.[6] The coefficient of oxygen absorption and the bubble size are of special importance. The effect of increased air flow rate on the absorption coefficient is of interest, and seems to limit the charge for systems that depend mainly on oxygen absorption as a function of the retention time of the bubbles. In the system described, the oxygen transfer, when the bubbles are formed and when the bubbles are leaving the water surface, dominates the oxygen absorption when the bubbles are rising in the water.[2] Experience also indicates that the suggested medium bubble aeration is more efficient at increased rate of aeration as compared with the fine-bubble aeration of conventional systems.

The bubble size influences the absorption inversely as the function of d^3, and, accordingly, is of very great importance. It is possible to produce extremely fine bubbles (by mixing air and water under high-pressure) for use in the described system. However, since the production of very fine bubbles takes a lot of energy, it is necessary to use only a small amount of air, and it is doubtful if this method is comparable to the medium-bubble aeration when the oxygen requirement is high.

The shallow location of the aerator should make it possible to use deep aeration tanks with very little increase in the specific energy consumption. As mentioned, the oxygenation capacity of this system is mainly dependent on the input of air, and only to a small degree on the oxygen absorption during circulation and at the water surface. Since the friction loss at the straight parts of the circulation channel (Fig. 10) is very small, the required energy increase for circulation seems to be negligible.

EXPERIENCE FROM PLANTS WITH LOW HEAD AERATORS

Description of Plants and Purification Results.—The aerator is a new addition to the dispersed air systems in use. The first plants built with the low

head aerator that are now in operation are designed for low, conventional oxygenation rates. They are not designed with respect to later investigations regarding most efficient distributor depth, hole spacing, partition wall size, importance of turbulence, etc., or to the developments in the activated sludge process towards a higher specific loading. Furthermore, both the pilot plants and the first full size plants receive sewage of a very special composition, with a high amount of industrial wastes of unusual quality, such as tannery wastes, metal industry wastes, mineral and vegetable oil, wastes from the detergent and plastic industries, and so on.

Experience has been gained from the pilot plants at Mölndal and Hallsberg and more than 20 full size plants in Sweden including those at Enköping, Kumla, Växjö, Töreboda, Uppsala, Örebro, Norrköping and later from some plants abroad. Comparative tests have been carried out at a pilot plant in the city of Kassel, Germany.

The pilot plant at Mölndal is a step aeration plant. It indicated very high BOD reductions (up to 99%) and oxygen residual (6 to 8 mg per 1) at ordinary

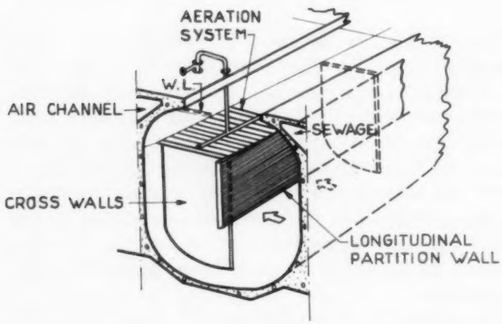


FIG. 9.—LOW HEAD AERATOR DESIGN

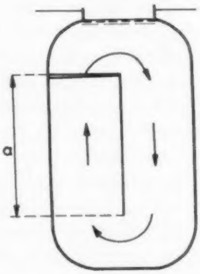


FIG. 10.—STRAIGHT PARTS OF CIRCULATION CHANNEL

sludge concentration and detention time. It receives sewage with acids and alkalies (pH 1.5 to - 11), vegetable oil, naphthalene, muriatic acid, and so on, which cause temporary drops in the efficiency, but the process very soon recovers. There is no trouble with clogging, although the plant is shut down occasionally.

At the pilot plant at Hallsberg, tannery sewage with a settled BOD of 1,200 was successfully treated to a very low BOD residual (10 to 30 mg per 1). The results from the activated sludge plant were continuously better than those from a similarly driven plant with couble alternating trickling filters, expecially during the winter.

The Enköping plant (May, 1953, population equivalent 52,000), which is the first full size plant, receives very concentrated sewage from metal industry, slaughter house, hospital, and military camps. It utilizes the conventional step aeration process. As the receiving stream is small, the plant was designed to deal with great variations in sewage flow. It has a very high efficiency, mostly

97% to 100% BOD reduction, though the sludge concentration is kept low (700 ppm to 1,000 ppm).

At Kumla (February, 1954, population equivalent 32,000) the recipient is a ditch running through cultivated areas, and the authorities required a very high bacterial reduction even at storm water flow. The step aeration plant is, therefore, over-dimensioned at average flow and some difficulties exist with the sludge because of overtreatment.

TABLE 1.—RESULTS AT VÄXJÖ

Sewage flow, average, cu m per day	7,250
Settled water, BOD5, average, mg per l	186
Effluent, BOD5, average, mg per l	18
Reduction of settled water, in percent	90
Load, BOD 5 per cu m-day, kg per cu m-day	1.42
Mixed liquor, sludge conc. DS, g per cu m	2,000
Load rate, BOD 5 per D S	0.7
Air, average effect, kw	21.0
D per o per capita, w per p	0.54
D per o per removed BOD, kwhr per kg BOD	0.37
D per o per removed BOD, kwhr per lb BOD	0.17
Oxygen, daytime 2 - 5, night 5 - 9 mg per l	

TABLE 2.—RESULTS FROM ÖREBRO

Sewage flow, average, cu m per day	31,000
Aeration basin volume load cu m per cu m/day	4
Settled water, BOD 5, average, mg per l	195
BOD5 applied per 24 hr, ton per d	6.05
Effluent, BOD 5, average, mg per l	13
Reduction of settled water, in percent	93
BOD volume load, kg per cu m-day	0.8
Mixed liquor, sludge cone, g per cu m	1,000 - 3,000
Energy consumption, kwhr per day	3,000
D per o per BOD5 removed, kwhr per kg BOD	0.55
D per o per BOD5 removed, kwhr per lb BOD	0.25

At Vãxjõ (August, 1954, population equivalent 38,000) a "full load" test has been carried out by disconnecting of half the plant. From a period in the spring of 1956 the results indicated in Table 1 were obtained.

Õrebro (May, 1957, population equivalent 173,000) is a step aeration plant designed for complete purification (about 95%) with regard to the small recipient. It receives the latrine from the unsewered part of the city, which exerts a severe load because of its bad quality. The design aeration detention period (excluding return sludge) is 4.5 hr. No investigations have been carried out regarding the most efficient operation. The results indicated in Table 2 are from a period in the spring of 1958.

During most of the day the oxygen content is above 5 ppm to 6 ppm. The distributor depth is only 560 mm. From these observations it is computed that it is possible to save more than 30% air and power by a slightly greater depth (800 mm) and closer control of air.

FIG. 11.—STEP AERATION PLANT AT UPPSALA

FIG. 12.—PREAERATOR AND DEGRITTER

Uppsala (July, 1957, population equivalent 180,000) is a step aeration plant (Fig. 11) designed for a high degree of purification (about 95%). As in Örebro, the recipient is a small river and latrine is added to the sewage. Furthermore, Uppsala is a very highly industrialized city with wastes that are difficult to process. The sewage system is separate, but leakage or connections to the storm sewer system exist. During the thaw period the average detention (excluding return sludge) in the aerating basin is less than 1.8 hr (volume load 13 cu m per cu m-day), but will rise to 5-6 hr in dry weather. The plant is operated with a low mixed liquor concentration and the oxygen content is ordinarily too high. It is not quite clear if the BOD determinations are occasionally influenced by waste from a drug factory. The BOD valve shows a great variance. With these reservations some results from recent operation are given in Table 3.

Clarification basins in Uppsala are 2-story rectangular basins. The load has been 0.8-1.3 m per h on the average per day. The basins seem to function well.

Saltsjöbaden and Nora have smaller plants (5,000 to 10,000 inhabitants) for complete treatment of domestic sewage mainly. The first one has a 3-story primary and secondary sedimentation tank and the second has a 2-story final

TABLE 3.—RESULTS AT UPPSALA

Sewage flow, average, cu m per day	40,000
Aeration basin volume load, cu m per cu m-day	4.6
Settled water, BOD 5, average, mg per l	160
BOD 5 applied per 24 hrs, ton per d	6.4
Effluent, BOD 5, average, mg per l	20
Reduction of settled water, in percent	87
BOD 5, volume load, kg per cu m-day	0.64
Mixed liquor conc., g per cu m	~1,000
Load rate, BOD5 per D S,	0.64
Energy consumption incl. sludge return pumping, kwhr per d	3,900
D per o per BOD5 removed, kwhr per kg BOD	0.61
D per o per BOD5 removed, kwhr per lb BOD	0.27

sedimentation tank. Both plants give very clear effluents with a high degree of purification (96% to 99%).

Töreboda is a small plant for a population of 5,000 but not for a comparatively large industry (milk and metal industry wastes). It is designed to treat dry weather flow completely and storm water flow by partial biological purification (60% to 70% B S reduction). The purification is very good and the sludge is thick and readily digestible.

Among plants recently put into operation are those at Norrköping (population equivalent 290,000, Fig. 12 shows the preaerator and degritter, Fig. 13 the aerating basin) for complete treatment with 3.5 hr detention time and 3-story final sedimentation tanks, and at Södertälje, (population equivalent 100,000), with no presedimentation, 30 min aeration and 1-story final sedimentation for partial biological treatment of a complicated domestic and industrial sewage (metal industry, drug factories, etc.).

In Kassel, Germany, a test plant consisting of three different activated sludge units equipped with porous plates, iron-stave-roters, and the low head aerator has been run with partial activated sludge treatment. Though the aeration basin

with the perforated pipes had an unfavorable shape and consisted of a short one-chamber basin, a satisfactory result was obtained. V. der Emde[10] reports similar effect and economy for the three systems under the conditions tested.

Clogging of the Air Dispersion System.—Though the aerator can be made up of porous tubes or, similar, all aerators hitherto consist of perforated pipes. The Sub-Committee on Air Diffusion in its excellent report[11] has only a short reference to perforated pipes as air distributing devices, stating that existing types are not very economic and are subject to clogging. However, where the clogging tendencies are exceptionally strong they might be preferable to porous media because they are fairly simple to clean and to restore. In a condensed list several cases of perforated or slotted systems are mentioned which have been abandoned because of too rapid clogging. The clogging has consequently been watched closely at the plants in operation.

FIG. 13.—AERATING BASIN

Clogging of perforated pipes from the air is no problem if simple precautions are taken. The clogging from the liquid depends upon its character. Aeration of sewage is used for several purposes, such as at oil and grease separation in a preaeration unit, or at aeration in activated sludge plants without presedimentation. Sometimes it is used to aerate effluent and sometimes to aerate the sewage in intermediate treatment, as before trickling filters. The sewage might contain suspended coarse matter or fine, suspended, and dissolved matters of organic origin, salt, hardness, iron and so on.

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10 "Beitrag zu Versuchen zur Abwasserreinigung mit Belebtem Schlamm," by W. V. Der Emde, Hannover, 1957.

11 "Air Diffusion in Sewage Works," FSIWA, Manual of Practice No. 5, 1952, pp. 4, 11, 15.

When nonsettled sewage is aerated with the grid type aerator, it has been found that exterior clogging can be important. The experience indicates that a very short-time aeration of ordinary sewage is possible without excess difficulties, but long time aeration of sewage with a high content of rags give rise to a spinning action with the effect that long ropes mat the grid and interfer with the water circulation, hence causing sludge to accumulate on the bottom of the aeration tank. At most types of efficient aerators of the wide band type, an unscreened sewage must be avoided. Fine screens are not more expensive than cutting devices in the sewage and actual removal of coarse material and rags are necessary, and economical also, in making possible a simple and efficient gritwash, an efficient inlet distribution in the sedimentation tanks and to avoid sludge deposits in big canals or pipes with wide variations in the waterflow. However, in some cases it is desirable to aerate crude sewage. In most cases especially at sludge recirculation plants there will be at a certain concentration of rags, big textile balls, and ropes even from finely cut material, which sooner or later mat the grids. Studies have disclosed that the shape and the material of the tubes have influence on the clogging. The exterior clogging seems to be completely avoided by special shaping of the air distributor. The oxygenation economy is maintained after a slight initial loss.

When presettled sewage is aerated by perforated pipes experience has indicated a normal interior clogging. Thorough investigations made by L. Malm, and Nordstroem, have disclosed, that it is the bursting of the bubble (about 10^6 times a day) upon leaving the hole in the pipe that caused sewage to enter and dry at the inside of the hole and pipe and thus clog it. Later studies of the interior clogging problem, in both laboratory and field, have been made. The clogging seems to depend upon such factors as, the air velocity through the hole, the shape of the hole, the direction of the air jet, and the concentration and character of the suspended or dissolved matters. These studies have indicated that some washing methods may increase the clogging, while the proper handling of the washing eliminates even a fairly severe clogging. The best way to avoid clogging is to remove deposits at once, that is, to maintain the cleanliness of the tubes. Some kinds of clogging as by iron deposits, seem to be removed by chemical means only.

Experience and investigations have shown, that under the circumstances studied, aeration by perforated pipes is practically possible, with very small costs for labor and other cleaning expenses. A great number of plants have now been operating for several years, with little difficulty with clogging, although the sewage from the Swedish communities contains a large amount of industrial waste.

Clogging of the holes in the pipes in preaerators can be more severe than in activated sludge aerators. The frequency of cleaning depends on the character of the sewage. At one plant a cleaning once a year has been desirable, whereas at others no cleaning has been necessary in a much longer time.

Inpsection and cleaning of the pipes is easily carried out, because they are arranged in sections of suitable size and connected to the main aerator tube by a flexible pipe. The "grid" is very light, its weight being about 50 lbs, and its shallow location simplifies removal and mounting.

Cold Weather Experience.—The cooling action of the air has given no trouble in the basins. Screens on the suction side of the fans have been clogged by ice in one case, due to the fact that they were located just above the near-by water surface of the aeration tank. Such location should be avoided.

The use of extremely high amounts of air at high loaded activated sludge plants, as 100 to 200 volumes of air per 1 volume of water, will cause no difficulties at ordinary sewage temperature, or even at very low air temperature, partly because of the raised air temperature by compression.

Foam From Detergents Caused By Coarse Bubble Aeration.—The coarse bubbles can separate from the water with less difficulty than very fine bubbles. Nevertheless very large amounts of coarse bubbles seem to create excess foam in about the same amount as fine bubbles with equivalently less air. Foam is, consequently a problem also with this type of aerator. At high oxygenation capacities foaming is a serious problem. Foaming is reduced in very dry weather and eliminated in rainy weather. In climates with high humidity and foggy weather foaming is especially severe. A concentrated sewage has greater foam-tendencies than a diluted sewage.

Foaming is reduced or sometimes eliminated by efficient control of the activated sludge process, especially of such factors as concentration of solids and oxygen.[12] It seems as if a suitable water spray has never failed to efficiently eliminate foam. A subsurface perforated pipe with jet dampers has been found appropriate in cold weather. Anti-foam agents may reduce the spray water, and can be recommended in difficult cases.

Circulation Rates and Sludge Circulation.—Since the air is distributed over a wide area and the partition wall introduces a certain resistance, it was expected that the velocity would be fairly uniform, and that the peripheral velocity would be less than in the common spiral flow aerators with a concentrated side aerator. Also, the turbulence was concentrated to the upper aerated zone. Some doubt was expressed regarding the possibility of circulating the sludge.

With a good air distribution however, the pumping action by this wide band aerator is considerable. Measurements indicate peripheral velocities near the bottom and near the wall in the rising section from 0.7 to 1 m per sec (3 to 4 ft per sec) or even higher. Experience has proved that every geometrical cross section of an aeration tank requires a certain minimum aeration level in order to keep the sludge in circulation. However, even old accumulations of sludge can easily be removed by raising the aeration level.

The aerator has been tested in preaeration tanks for washing and removal of grit. Good removal and a good washing can be effected. When aerating a heavy sludge a check of the circulation rates should be carried out.

Influence of Turbulence on the Rate Purification.—It has been assumed that dispersing the biological floc in very fine particles would facilitate the oxygen absorption and increase the rate of purification.[13] Comparative tests at Kassel, between this aerator, porous plates, and rotor aerator have indicated no noticeable difference in this respect. It is possible that none of the aerators cause a remarkably fine dispersion, or perhaps the oxygen passage is sufficient in the floc of the ordinary type. At a high turbulence in the water the soft floc particles will change shape and thereby pump water in and out of the particles. A too complete dispersion of the floc has obstructed reflocculation, caused decreased clarification[14] and must be avoided.

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12 "Cause and Control of Detergent Frothing—A Review," by Benn Martin, SIWJ, November, 1954, p. 1413.

13 "Research on Activated Sludge III, Distribution of Oxygen in Activated Sludge Floc," by A. Pasveer, SIWJ, Vol. 26, p. 1.

14 Manchester Rivers Department's Report, March 31, 1956.

Sanitary and Safety Aspects.—Aeration of water may cause infection of the air near the aeration tank. At covered plants changes in the air composition will occur that might be dangerous.

Oxygenation by spreading drops into the air by mechanical means is more effective the finer the drops are, but at a certain size the drops might easily be carried up in the air causing infection. At all types of compressed-air aeration as well as at trickling filter distributors a similar effect might be caused by splash of the drops or the bursting of the bubbles at the water surface. Also the use of spray-water to eliminate foam may cause infection of the air. Repair work immediately above the water surface should be avoided with the aeration system operating. At the low-head aerator investigations indicate a height of less than 1 ft will suffice for preventing the platforms of being wet. The risk for infection seems therefore to be very small, even at a high aeration level. There are over 30 plants in operation, some of them covered plants, and no case of infection is known.

During aeration of activated sludge, oxygen is absorbed from the air and at the same time carbon dioxide is produced. For every pound of oxygen absorbed about 1 lb of CO_2 is released from the water. Every cubic meter (35.3 cu ft) of air contains about 0.2 kg (0.45 lbs) of oxygen. Then oxygenating 1 cu m of activated sludge-sewage mixture by a mechanical aerator in an unventilated covered plant the oxygen in 1 cu m of air will not be sufficient longer than 1 hr at a modern, high oxygenation process. At the same time as the percent oxygen is reduced, the percent carbon dioxide is raised, and the air in a covered plant will consequently be dangerous in a very short time. A turbine aerator or an injector aerator with a high absorption efficiency (30% to 40%) will reduce the oxygen to 12% to 14% and raise the carbon dioxide to 5% to 6%. The ventilating system must, therefore, be powerful and absolutely reliable with such aerators. With "diffused air" (porous plates or tubes) the reduction of oxygen (to about 17% to 18%) and increase of carbon dioxide (to 1.7%) will cause great risks, especially because the exhaust air sometimes contains other gases as small amounts of hydrogen sulphide and so forth. The low head aerator reduces the oxygen in the air only to 19.5% and produces carbon dioxide to 0.35% and is, therefore, more or less self-ventilating.

Some mechanical aerators are very dangerous to the operating personnel. The aerator under discussion is in itself safe, but water circulation rates can be high and normal care should be anticipated under operation.

Noise.—At a central location of a purification plant freedom from noise is important. The fans for a low-head aerator, and the suction pipes, cause less noise than the compressors for ordinary "diffused-air" plants. Even a simple housing of glass walls reduces the noise to a very low degree. A very efficient reduction can be obtained by placing the fans in a concrete structure below ground. Of course the selection of fans and motors is of importance, as is the shaping of the suction pipes and air intake.

OXYGENATION DETERMINATIONS ON TAP WATER AT THE NACKA STATION, STOCKHOLM

Purpose and Methods.—It is not possible to investigate the capacity, economy or limitations of an aeration system by studying operation-records from actual plants. The designing engineers or the communities are not willing, or

allowed, to take great risks when building and operating new plants, and furthermore, in this way new ideas could be tested first after many years have elapsed. The quality of sewage will influence the results considerably. A much more rapid development is possible by testing the aeration system - as now is

FIG. 14.—PILOT PLANT AT NACKA

customary - in pilot plants with pure (tap) water. Also, the system described has only been tested since the summer of 1956 in such a plant.

The pilot plant at Nacka outside of Stockholm (Fig. 14) consists of an one meter long medium sized section of an activated sludge aeration channel, the

wet cross-section being 3.2 m-by-3.2 m (10.5 ft-by-10.5 ft). Fig. 15 shows a distributor pipe in use.

The oxygenation determinations have been carried out on deoxygenated water as described by Kessner and Ribbius,[15] Pomeroy,[16] Wuhrmann,[17] Pasveer,[18] and others, and have been recalculated to be valid for aeration against zero-oxygen at 10°C and 760 mm Hg in the water during the entire test period.

All tests are carried out with pure water (tap water from the Stockholm Water Works) in the basin. The pure water "liquid coefficient" is consequently assumed to be 1.

To remove oxygen present in the water sodium sulphite (Na_2SO_3) is used with cobalt chloride ($CoCl_2$) as a catalyst. The water is intensively mixed with the dissolved chemicals as the basin is filled up. The aeration is started at the rate of air, depth of distributor, and so on to be tested, against zero-oxygen at the time 0. At short intervals oxygen determinations are made (by the Winkler method) until saturation is reached. A typical dissolved oxygen curve is shown in Fig. 16. The oxygenation capacity is computed from the equation

$$O_{CL} = C_S \frac{p + 760}{760} \frac{k_{10}}{k_t} \frac{1}{T_2 - T_1} \ln \frac{C_S - C_1}{C_S - C_2} \qquad (14)$$

in which T_1, T_2 are the time at two different moments, t is the temperature, k_t indicates the oxygen uptake coefficient at temperature, t, °C, k_{10} denotes the uptake coefficient at temperature 10°C, C_1 is the oxygen dissolved at time T_1, C_2 is the oxygen dissolved at time T_2, C_S is the oxygen saturation at temperature t, p denotes the pressure, representative for the depth of air distributor, and O_{CL} refers to the oxygenation capacity at specific air rate L and other influencing factors.

Since these tests on tap water are carried out in a basin without throughflow of water, the rate of air dispersion is expressed as the volume of air per hour (Q_A) at the actual depth, h, of the distributor divided by the volume of water in the basin (Q_B) instead of by the water flow as volume of water per hour. This factor $L = Q_A \; Q_B$ is here called the aeration level (h^{-1}) and must not be taken as the amount of air per volume of water treated (specific air-consumption). The latter figure will be obtained by multiplying the former by the average detention time. So if there is an aeration level of 10 and 2 hr detention time, the amount of air used up will be 20-volumes-per-1-volume-water-treated. The aeration level at a distributor depth of 1 m will be called the specific aeration level L_S.

Factors Studied.—Investigations have been carried out regarding the influence of the "oxygenation capacity" of (1) depth of air distributor, (2) distribution of the air, (3) rate of air dispersion (size of aeration level), (4) width of air distributor, (5) shape of basin, and (6) size of partition wall, depth of aeration tank. The influence of the size of holes or the use of porous tubes or plates or special devices, such as causing increased diminishing of the bubbles and the floc has not yet been investigated.

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15 "Comparison of Aeration Systems for the Activated Sludge Process," by H. J. N. H. Kessener and F. J. Ribbius, SWJ, Vol. 6, No. 3, May, 1934, pp. 423.

16 "Power Efficiency in Activated Sludge Aeration," by R. Pomeroy, SWJ, Vol. 20, No. 2, 1948.

17 "Ergebnisse von Grossversuchen an hochbelasteten Belebtschlammanlagen und Tropfkörpern," by K. Wuhrmann, Die Schweiz. Z. Hydrol, 15, 1953, p. 1.

18 "Research on Activated Sludge, I and II," by A. Pasveer, SIWJ, Vol. 11, No. 12, November, December, 1953.

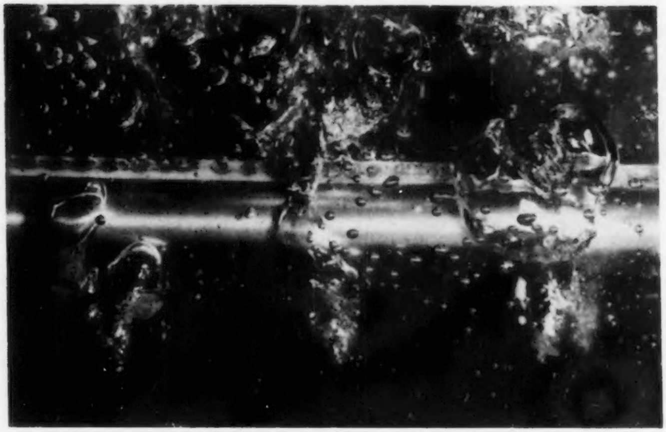

FIG. 15.—DISTRIBUTOR PIPE IN USE

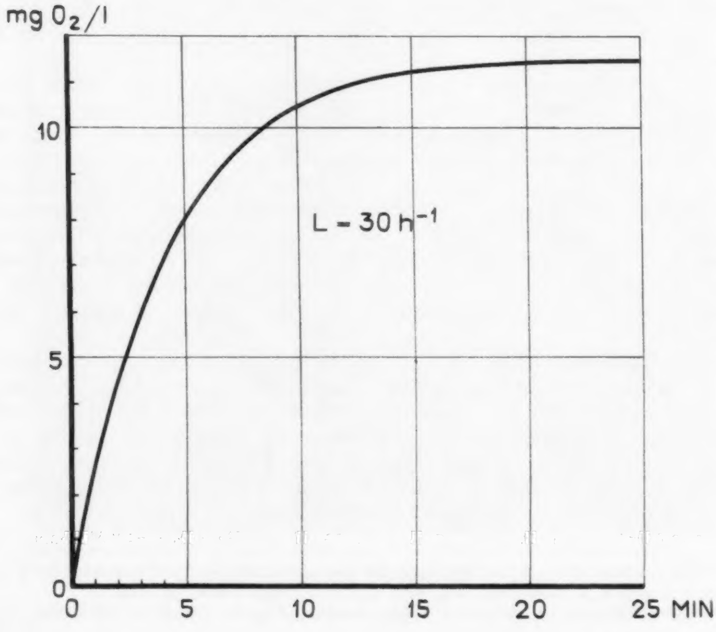

FIG. 16.—TYPICAL DISSOLVED OXYGEN CURVE

The majority of tests have been carried out at the conventional, fairly low, oxygenation rates of about 30% to 90 gr O_2 cu m hr. A limited number of tests have been carried out at greater oxygenation rates. At the very high oxygenation rates obtainable with this system certain difficulties arise when making the necessary determinations. Special investigations have been carried out regarding the hydraulics of the aerator.

Oxygenation Capacity.—Results on oxygenation capacity from representative test series are given in Table 4 and Figs. 17 through 23. Fig. 8 illustrated the course of oxygenation at different depths of air distributor under two different primary assumptions. In the first case [Fig. 8(a)] it is assumed that the aeration level is constant at all depths. The relative effect is computed as a straight increasing line (at constant blower efficiency) and the relative oxygenation is experimentally found to be more or less a similar straight line. In the second case [Fig. 8(b)] it is assumed that the input of electric energy is constant at all depths. The aeration level is then inversely proportional to the depth. The relative oxygenation is experimentally found to vary, as the curve on the diagram indicates. The results of the investigations under the two different assumptions are shown in more detail in Figs. 17 and 18.

Fig. 17 shows the case when the aeration level is kept constant and the distributor depth varied. The effect and the total oxygenation increase as straight lines with the depth. The relative oxygenation is fairly constant with the depth, though a slight increase can be observed with the depth. Fig. 17, which also shows the oxygenation at different depths of the air distributor at constant aeration level within a narrow range, indicates that the oxygenation at constant dispersion rate (constant aeration level) increases more or less in proportion to the distributor depth, as Ippen and Carver[6] report. This also seems to be in accordance with investigations by King.[9] He has found the influence of depth to be

$$O = K\,H^X \qquad (15)$$

where X is 0.71 at 5% plate area and X = 0.77 at 10% plate area. At 50% diffuser area X might approach 1.

In the case shown in Fig. 18 the effect is kept constant and the aeration level varies inversely with the depth. At L_S = 5.7 the total oxygenation increases with the depth to about 700 mm (2 1/2 ft) but decreases at greater depth. The effect is thereby constant about 190 W. The relative oxygenation increases as the total oxygenation from zero at zero depth to a maximum at the said depth of 700 mm, after which it decreases fairly sharply. Similar curves have been obtained at other effects. This course indicates that at a given effect it is considerably more economical to use, say, twice as much air at 750 mm depth as at 1,500 mm depth, and also than, four times this amount at 375 mm depth, because there is, in the investigated case, a certain depth of about 750 mm that gives maximum economy. An explanation of this might be that the increased amount of air at decreased depth will shift the influence of contact time to influence from newly produced air bubbles, will also increase the turbulence, and hence cause a more rapid exchange of water around the air bubbles, and perhaps a dispersion of the bubbles into finer ones. All of this contributes to an increased over-all oxygen absorption coefficient. At very low depth the retarded circulation causes an over-aeration (loss of air) and a decreased total oxygenation, as indicated by the curve.

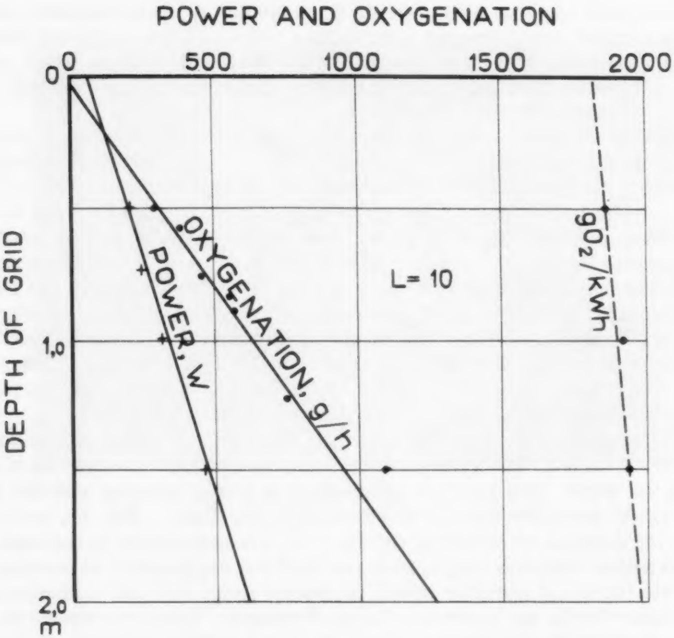

FIG. 17.—RELATIVE OXYGENATION FOR CONSTANT AERATION LEVEL AND VARIED DISTRIBUTOR DEPTH

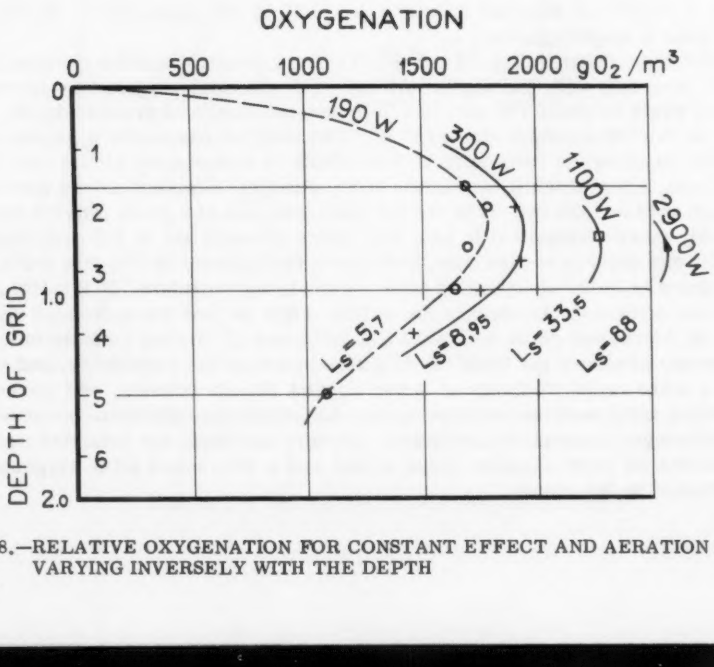

FIG. 18.—RELATIVE OXYGENATION FOR CONSTANT EFFECT AND AERATION LEVEL VARYING INVERSELY WITH THE DEPTH

Experiments have shown that the course and shape of the curve changes at increased specific aeration level and shape of waterway. At very high aeration rates the depth for best oxygenation increases slightly and the decrease of oxygenation at greater depth is not so great as at normal aeration rates, because the turbulence will still be sufficient to produce a good absorption coefficient at the reduced aeration rates.

The oxygenation curve of Fig. 18 indicates the importance of determining the most efficient aerator depth for the actual local conditions. A similar effect to that described is reported by King.[9] For porous plates King got a maximum oxygenation at about 750 mm, a rapid decrease below 750 mm and a slow decrease above this depth. In this case, however, the depth is tank depth, and "diffusers" are located on the floor. A low aerator depth and air pressure makes it possible to use fans, which give a better blower efficiency and are mechanically simpler than medium-head compressors.

Fig. 19 indicates the influence of air distribution. The diagram shows the oxygenation at different relative hole number per unit area, and the great importance of a sufficient closeness of holes in the aerator to obtain low power consumption As it is difficult to supply enough holes at an aeration rate which is too low, the necessity of using a fairly high air dispersion rate - and consequently, high specific load - is stressed, in order to get the best economy with the aerator.

Fig. 20 shows the relation between the oxygenation capacity in grams per cubic meter-hour and the aeration level at different depths of the distributor. Since the absolute oxygenation values depend upon the type of distributor and other factors, correct values will be obtained by multiplying with a factor C, determined for the distributor to be used. Only a few determinations have been made at very high oxygenation values, and the curves are somewhat uncertain in that range. The oxygenation capacity seems to increase in slightly accelerated proportion to the air dispersion rate. Very high oxygenation of about 1,200 gr per cu m h has been obtained. No upper limit has been found. Earlier investigations seem to indicate a decreasing oxygenation rate with increasing rate of air dispersion.

This probably depends on the fact observed by Ippen and Carver[6] that for a given nozzle the oxygen absorption is higher for lower gas flow rates than for higher ones. Tests with porous plates are usually carried out with constant plate area, whereas these tests with perforated pipes are carried out with approximately constant air flow per hole. Increased turbulence might also be an explanation.

The relative width of the aerator has an influence both on the capacity and the economy of oxygenation. As seen from Fig. 21, the core that does not pass the aerator increases at decreasing relative width. It is of importance that the core moves faster than the water near the walls.[19] The mixing between the aerated "cylinder" and the core increases, with increased relative width of aerator. The result of some tests is shown by Fig. 22.

Of importance, also, is the over-aeration of the narrow aerated part and the under-aeration of the core, which will increase the risk of oxygen deficiency during severe operation.

The influence of the shape of the basin has been studied mainly in respect to the need of deflecting or rounded corners. It is evident that deflecting corners impart economy. At low aeration levels they might increase the effect as much at 15% at high aeration level they can be detrimental.

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[19] "Sewage Treatment," by Imhoff and Fair, New York, 1956, pp. 140, 141.

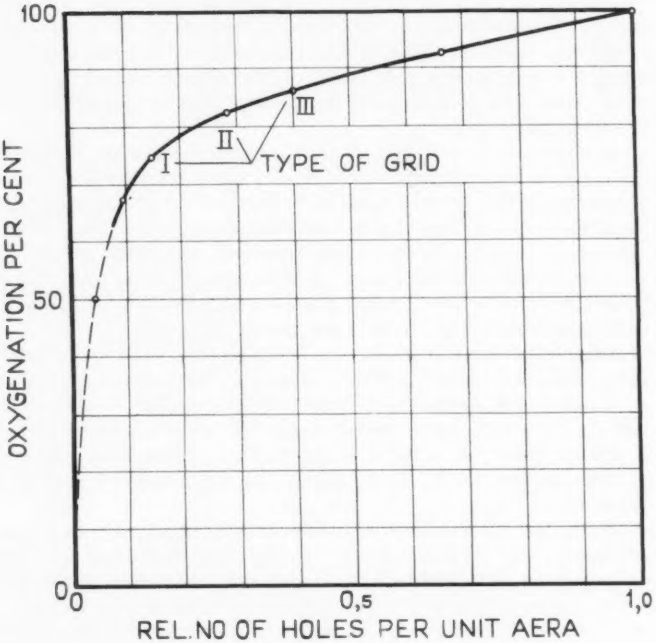

FIG. 19.—INFLUENCE OF AIR DISTRIBUTION

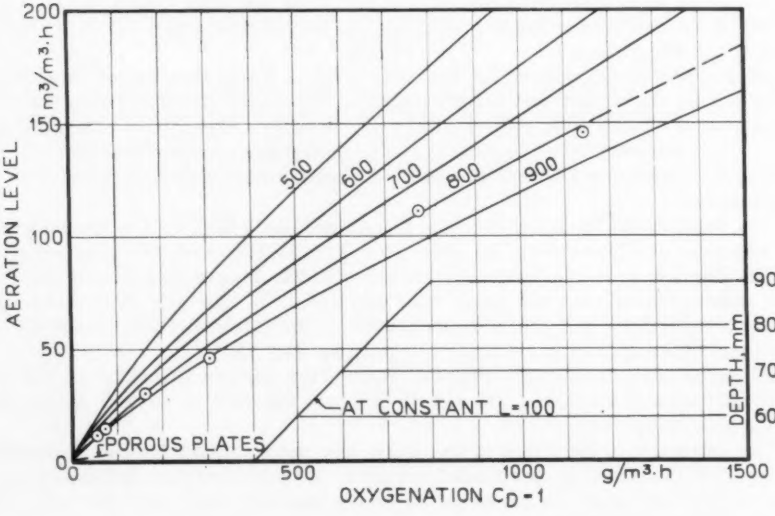

FIG. 20.—OXYGENATION AT DIFFERENT AERATION LEVEL AND DISTRIBUTOR DEPTH WITH STANDARD DISTRIBUTOR

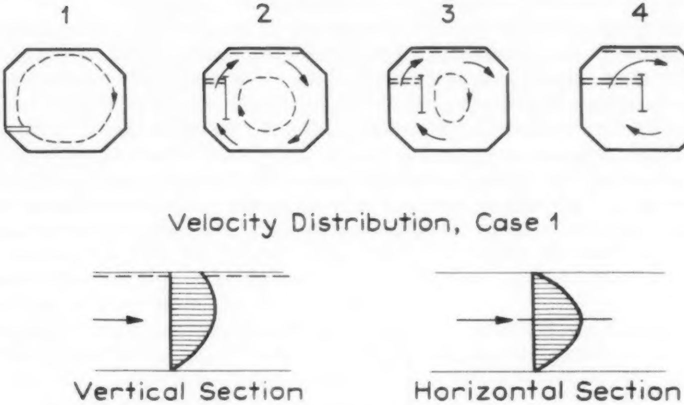
1
2
3
4
Velocity Distribution, Case 1
Vertical Section
Horizontal Section

FIG. 21

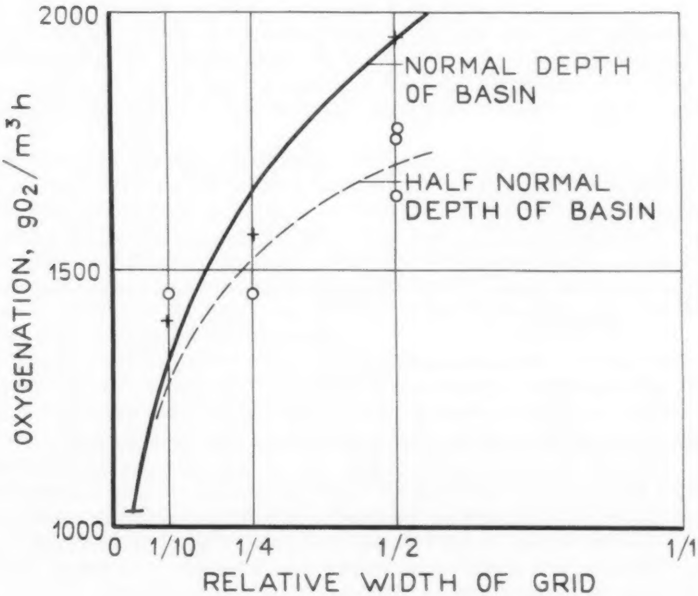
2000
1500
1000
OXYGENATION, gO_2/m^3h
NORMAL DEPTH OF BASIN
HALF NORMAL DEPTH OF BASIN
0
1/10
1/4
1/2
1/1
RELATIVE WIDTH OF GRID

FIG. 22

The size of the partition wall has been the subject of some tests. The removal of the partition wall always increases the consumption of power, even in shallow tanks, where the effect was expected to be negligible.

Fig. 23 illustrates the fact that variation of the depth of an aeration tank does not influence the absolute capacity of an oxygenation system located in the upper part of the tank. Consequently, the specific oxygenation increases in inverted proportion to the depth. This fact is in accordance with Pasveer's observations[20] with rotor (brush) aeration. But it is important to make clear the difference between oxygenation by air dispersion and by the mechanical surface aerators. By means of air dispersion the specific oxygenation can be increased by not only decreasing the tank depth but also by increasing the air dispersion rate (at constant or increased depth). Consequently, it is possible

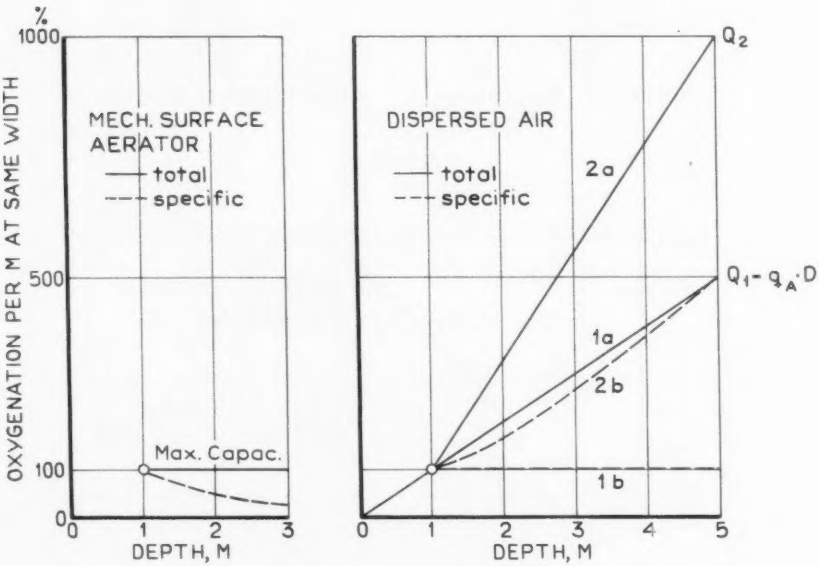

FIG. 23

to obtain such high specific oxygenation rates as are obtained with mechanical surface aerators in very shallow basins and in very deep basins with air dispersion. If the amount of air is increased in proportion k times the depth H, the total and the specific capacities, respectively, will be increased by (k H H) and (k H/H) times the depth. A true bubble layer (see Fig. 4) is developed first at aeration levels above 1,000. It is known that aeration in activated sludge is quite possible technically and economically with bubble-layers.

Circulation Rates.—Extensive investigations have been carried out regarding the hydraulic properties of the aerator. The bottom and rising velocities

20 "Research on Activated Sludge, II," by A. Pasveer, SWJ, Vol. 25, No. 12, 1953.

has been measured and studied with regard to the subsidence, the arosion of sand of different size, and grading.

It is known that the mammoth pump at a high air-water mixture loses its efficiency. The "slip" of the airbubbles increase and finally the pumping effect reaches a maximum. When the pump circulates the water, a braking effect in the downstream section, added by the bubbles, will not release the water in the top section. No doubt these bubbles and their slip adds to the aeration and no decrease in the oxygenation has been found. In addition the circulation shows irregularities.

Generally it has been found that the circulation starts very forcibly and then slows down slightly. At increasing air rates the circulation increases up to a certain rate, above which it is more or less constant or might decrease. This depends upon the shape and size of the basin and especially such items as deflecting walls and the partition wall. The deflecting wall can increase the circulation as well as brake the circulation.

For good adjustments it is generally possible, at equilibrium, to get bottom and rising rand velocities of 0,6 - 1,2 m per sec (2 to 4 ft per sec), which is sufficient to erode fairly coarse sand. These velocities will be maintained, even at fairly low air rates.

The tests have been carried out in the Nacka test basin, where it is possible to build in basin section to full scale or to scale 1-to-2, even for the biggest basins. The velocities in the prototype may be computed according to the Froude model law. A pendulem, which has been seperately calibrated has been used in most cases to determine the water velocities.

It is concluded, from these tests, that every full size high capacity plant should be controlled and trimmed with regard to its water velocities. Preferably every plant would have a simple device indicating the water velocity at important points, in order to avoid sludge deficit in the water at some operating rates.

Absorption Efficiency.—Absorption efficiency is defined as the amount of oxygen absorbed in relation to the amount of oxygen supplied to the distributor. The absorption efficiency is of importance at oxygenation with pure oxygen, and perhaps at aeration in covered plants. The absorption efficiency will have an influence on the investment cost for piping, blowers, and so on. Still, the absorption efficiency can be a very misleading figure, because it has no relation to the capacity or overall economy, which are the factors of primary importance for the development of aerators.

Economic Considerations.—The investigations have been concentrated on the factors having an influence on the oxygenation capacity. Investigations regarding improvement of the economy are, however, being carried out continuously. It has been found that from time to time great improvements in the economy have been possible.

Economy figures are given in several different ways, which can be very confusing and sometimes misleading. Figures such as pounds of oxygen introduced in the water per 1 k hr might be computed per net energy (without losses) or per gross energy (with losses in a blower, in bearings, transmission, gears or speed variators) or as motor input energy. Energy should be computed for necessary equipment as motor axis brake energy. for best conditions as efficiency on blowers, gears, and so on. It is of importance to know if oxygenation is given for a small basin or a full size basin. Only the latter ought to be accepted because the turbulence is relatively too high in the smaller basins. Such a value is suggested as "the oxygenation economy index" (at a specified testing method). It is only a primary figure however. To get a figure for practical

purpose the oxygenation economy index I_o must be multiplied by a coefficient k, for actual operating efficiency (as influenced by such factors as variable load, water level) and another k_2 for influence of decreasing efficiency (wear, clogging and so on). Hence,

$$I = k_1 k_2 I_o \qquad (16)$$

It should now be possible to obtain tenders for aerators on a guarantee for the specific oxygenation and the motor energy per pound of oxygenation tested on pure water at a specified method. The test should be made in a section of the actual basin with sufficient length.

For the low head aerator with perforated pipes and medium sized holes investigations indicate, at conventional low capacities, an oxygenation economy index of about 1.8 kg to 2.2 kg O_2 per k hr (4 lbs to 5 lbs per k hr), at medium capacities 2.5 kg to 3.1 kg O_2 per k hr (5.5 lbs to 7 lbs per k hr) and at very high capacities about 3.5 kg O_2 per k hr (8 lbs per k hr), determined with the sulphite deoxygenation method at 10°C, zero oxygen and 760 mm barometric pressure. These values are determined for a section of up to 10 ft by 15 ft (volume about 5 cu m to 13 cu m per meter of basin length). More expensive distributors can give higher economy index and may be motivated at a high kilowatt hour price. More research work is necessary, however, to confirm these data and to investigate the economy index at very large basin sections.

It is of certain interest to compare this system to the classic one where porous plates at the bottom of a basin are used. According to results published by King[9] and Rodhe[21] it is seen that the oxygenation per cubic meter of air is higher for the clean porous plates than for the perforated pipes at the same aeration level, but the absolute capacity is very low. The specific oxygenation per kilowatt hour used up is superior for the new, clean, porous plates at very low aeration level, especially for the fine porosity plates, but is equal or inferior for higher aeration levels.

In actual operation we must consider the influence of the clogging and the volumetric efficiency. Investigations have shown that plants with porous distributors are often badly clogged and operate with about half economy. There are also well managed plants, which keep the clean-curve economy.

It is also of importance, with regard to actual operating economy, that porous material must always get a minimum amount of air. It is not possible to shut out aerating units at low load without emptying the basins or lifting the dispersers, and sufficient reserve in blowers and local power generators is necessary.

The figures for tap water at 10°C and zero oxygen cannot, of course, be taken as valid at activated sludge treatment of different wastes without necessary corrections. Even the figures on oxygenation obtained on the specific liquid in question may not be quite representative. Certain factors characterizing the aeration system (such as turbulence), the sewage (poisonous matter, and so on) and the process (amount of returned sludge, feeding) are of so great an importance that only tests will give reliable information in the local case.

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21 "Belüftungsversuche auf der biologischen Kläranlage Gevelsberg des Ruhrverbandes," by Rohde, Das Gas—und Wasserfach H, 52, December, 1957.

Of great importance is the "liquid coefficient". Though it is not satisfactorily investigated the following values might be given as a guidance:

Settled sewage, concentrated	0.4 - 0.5
Settled sewage, diluted	0.5 - 0.6
Active activated sludge	0.8 - 1.1
D:o, mixed liquor	0.5 - 1
Salt sea water	0.8

It is seen that an activated sludge plant needs about 100% more air to start than to run (or 50% of the sewage must be bypassed). The same is true if the activated sludge is inactivated during operation. A correction must be made for the minimum oxygen required in the liquid and due regard must be taken to such factors as variations in flow, BOD, temperature, and the future demand to get the purification required.

OXYGENATION DETERMINATIONS AT THE EAWAG IN ZURICH, SWITZERLAND

Purpose and Methods.—In order to confirm the Nacka tests and extend the investigations, especially in the high oxygenation capacity range, further research has been carried out at Tüffenwies, of the Eidgenössische Anstalt für Wasserversorgung, Abwasserreinigung und Gewässerschutz, at the Institute of Technology in Zürich, Switzerland, EAWAG, under the supervision of K. Wuhrmann.

The tests are carried out in a basin with a depth of 4.8 m (16 ft), a width of 3.2 m (10 ft), and a length of 4.9 m (16.5 ft). The basin and the air distributor have been finally adjusted according to the best experience from the Nacka test station.

Oxygenation has been determined on settled sewage and on tap water. In the first case settled Zürich sewage stands until zero-oxygen content is reached, at which point aeration is started and proceeds as with the sulphite deoxygenation method. In the case of tap water determination can be made by either, (1) the sulphite deoxygenation method using a copper salt as a catalyst, (2) the sulphite surplus method described by Kountz,[22] and (3) the ferrous-sulphate method, also using copper salt as catalyst. The surplus method gave higher values that the other methods, and those values were excluded. The results are given as the coefficient K_{10} in

$$O_{CL} = K_{10} C_{10} \qquad (17)$$

in which O_{CL} is the oxygenation capacity at actual aeration level, distributor depth, and so on, C_{10} denotes the oxygen saturation at temperature 10°C and p mm barometric pressure, and K_{10} is a coefficient. The coefficient K_{10} is a measure of the specific oxygenation of an aerating device. To get the economy index it is necessary to know the losses in friction, head, efficiency, and so on.

Tests with both settled sewage and pure water have shown that the ratio of aeration level to K_{10} is independent of the basin depth. That means that the specific aeration is inversely proportional to basin depth (or volume per meter of basin length), all other circumstances unchanged.

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22 "Biological Treatment of Sewage and Industrial Waste," by Kountz, Vol. 1, p. 213.

With presettled sewage from Zürich, which is fairly concentrated, the oxygenation was low and about 40% to 50% of the computed tap water values. The oxygenation was not proportional to the rate of aeration, but decreased noticeably with the air rate. The aeration channel was tested at 2.8 m (9.2 ft, 3.65 m (12 ft) and 4.8 m (16 ft). No difference in the economy was found between these limits.

The tap water tests confirmed the important relation between oxygenation and aeration rate found at the Nacka station. This relation was found at Tüffenwies at be a straight line. A slow accelerated increase of the oxygenation at Nacka may depend upon the fact that the number of distributor openings was increased with the air rate whereas they were constant at Tüffenwies.

The oxygenation found at Tüffenwies on deoxygenated water was considerably higher than at Nacka, about 20% to 30%, which was thought to be explained by the better type of basin (better cross-section, greater length) and distributor used at Tüffenwies. Later a crack in a welding was found at the comparable grid used at Nacka and this proved to be the main cause. Some results are given in Table 5 (800 mm distributor depth, 31 cu m basin volume).

A comparison of the results on settled sewage at Tüffenwies and on tap water at Tüffenwies and Nacka (after repair of the grid) is shown in Fig. 24.

GENERAL REMARKS AND CONCLUSIONS

Advantages From a High Oxygenation Capacity.—By modern activated sludge processes it is possible to vary the load considerably from complete treatment to any degree of partial treatment. The purification effect can be maintained at a considerable increase in volumetric load (reduced detention time), but decreases with the load expressed as amount of impurities in relation to weight of sludge in action in the aerator. Heukelekian, Okun and others have suggested the load be expressed as the rate of BOD applied to BOD carried in the plant (aeration basin).

Investigations at numerous plants[23] have shown that the activated sludge process alone (exclusive of primary treatment) will yield about 90% BOD removal if the load does not exceed 50 lbs per day per 100 lbs of suspended solids in the aeration tanks. When the BOD load rate is increased, the per cent reduction decreases. At a load of 100 lbs the reduction is figured at about 75% and so forth. Similar experience is presented by Wuhrmann, Von der Emde,[10] and others. Von der Emde suggests the load rate as BOD 5 applied per day to the value of D S in the aeration basin. For 90% purification he gives a load rate of 0.5 to 1.5 depending on the character of the sewage, and for partial treatment, a load rate of 1 to 2.5.

In a given basin the BOD (or D S) carried, depends mainly upon the concentration of suspended solids in the return sludge and mixed liquor. An increase in the capacity can, to a certain extent, be secured by increasing the volume of return sludge. This is preferably done by concentrating the sludge liquor. There seem to be several means of doing this, and consequently of increasing the load per unit volume economically and reducing the size of the aeration basins. But with very concentrated sludge liquors, mixed liquors, and reduced aeration time the claims on the aeration system will increase tremendously.

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23 "A Rational Approach to the Design of Activated Sludge Plants," by T. R. Hazeltine, W & SW, November, 1955.

These can be met however by modern aerators. With improved methods for clarification, sludge dewatering, and sewage aeration, there should be better prospects of great developments of the activated sludge process in the near future than ever before. High rate plants with a very short detention time must be designed for the great variations in the oxygen consumption of the sewage however, and this might be an economical limiting factor of too short a treatment time.

TABLE 5

Aeration level, in cu m per cu m-hr	Coefficient K_{10}	Oxygenation, in gr per cu m per hr	Economy index, in kg per kwhr
15	10	110	2.5
60	45	495	2.7
84	61	670	2.8

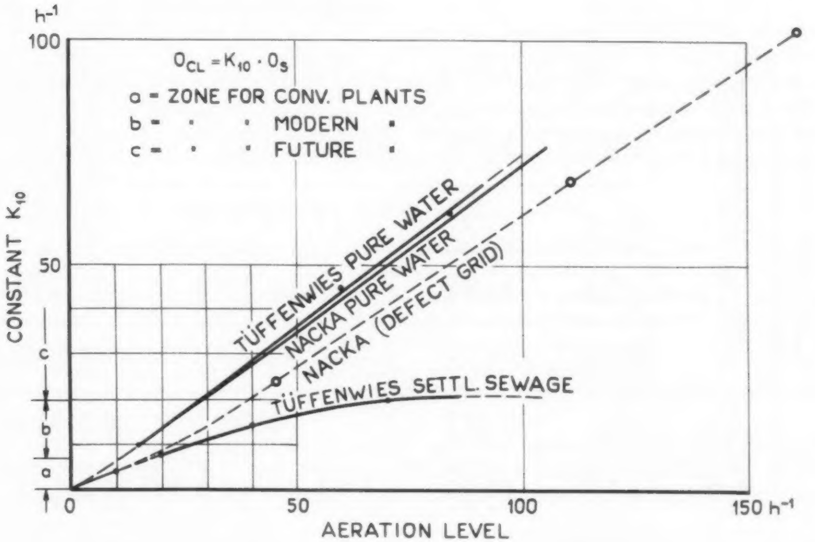
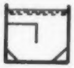


FIG. 24

At present the oxygenation rates obtained - in this case a tap water figure of about 1,250 per cu m-hr has been obtained in a 10 ft basin, which is about 25 times as much as the conventional rate of 50 gr per cu m hr - exceed the need in sewage practise, but might be useful in the fermentation industry. They can also be used for oxygenation of water only, as effluents, diluting salt water, and so on, in short time aerators. When the sewage is used for irrigation

TABLE 4.—RESULTS OF OXYGENATION CAPACITY TESTS

TEST NO.	TYPE OF BASIN	WATER		GRID		FREE AIR		COMPR. AIR		OXYGENATION			K_{10}	REMARKS
		VOL. m^3	TEMP °C	TYPE	DEPTH mm	TEMP °C	BARO mmHg	HOUR m^3/h	VOL. WATER $m^3/m^3 \cdot h$	TOTAL g/h	SPEC g/m^3h	SPEC g/kWh	l/h	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	
12		8.64	12.5	I	500	-	750	91	10.5	290	34	1560	2.9	Figures in column L-P are calculated at 10°C and 760 mm Hg.
22		8.96	14.7	I	500	14	752	58	6.7	202	23	1698	1.9	
11		8.96	12.2	I	600	-	764	90	10.0	385	43	1792	3.6	Figures in column N are calculated at $\eta = 0.8$ and losses equal to 100 mm H_2O
23		8.96	14.6	II	600	15	750	92	10.3	407	45	1859	6.2	
24		8.96	14.5	III	600	16	752	93	10.4	426	48	1919	4.3	
25		8.96	14.3	I	600	16	762	58	6.5	246	28	1770	2.3	
16		8.96	14.0	I	750	21	763	92	10.3	472	53	1773	4.4	
21		8.96	14.7	I	750	15	752	56	6.3	299	33	1845	2.8	
15		8.96	13.8	I	825	21	762	92	10.3	578	65	1992	5.3	
20		8.96	14.7	I	825	15	750	56	6.3	328	37	1862	3.0	
17		8.96	14.0	I	870	21	762	92	10.3	584	65	1919	5.3	
19		8.96	14.7	I	870	16	750	57	6.4	356	40	1883	3.3	
34		8.96	7.0	I	1000	3	770	86	9.6	599	67	1859	5.4	
35		8.96	5.5	I	1000	1	765	51	5.7	307	34	1634	2.8	
37		8.96	6.0	I	1220	6	758	88	9.85	755	84	1905	6.7	
32		8.96	7.0	I	1500	1	770	34	3.8	200	22	1081	1.7	
33		8.96	7.0	I	1500	3	770	90	10.0	1129	126	2299	9.8	
30		4.83	13.0	I	600	13	764	50	10.4	211	44	1773	3.8	
28		4.83	13.0	I	600	13	770	52	10.7	380	79	1767	6.6	
31		5.38	12.0	III	750	11	752	243	45.2	1580	293	2237	24.3	
18		8.96	14.5	II	700	21	760	268	30	1416	158	1938	13.2	
38		8.96	6.0	IV	800	- 2	776	996	111	7480	829	2591	68	
39		9.02	6.0	IV	800	- 3	780	1290	162	9890	1236	2500	102	

the excess sludge might be biostabilized and the sludge disposal plant perhaps eliminated. The nutrient in the sludge will be transported with the sewage for no extra cost.

SUMMARY AND CONCLUSIONS

The primary purpose of the development of the aeration technique using compressed air is for the reduction of air consumption and the production of fine bubbles. Reduction of air saves the energy consumption proportionally, whereas making bubbles smaller increases it rapidly. At present (1960) economy is generally expressed by a low air consumption rate. Oxygen absorption may vary with different aeration systems. Systems with a low per cent absorption might be underestimated. Air is abundant in infinite amounts and, consequently, it is not the air but the power per pound of oxygen introduced into the water that is of importance. The air-rate or air absorption efficiency must be abandoned as a measure of aerator economy and substituted by energy (as k hr) in proportion to oxygen uptake (as lbs per hr). An "economy index" computed as as to give unmistakable figures has been presented. All types of aerators can then get a primary factor of comparison. Of course, other factors must also be taken into consideration.

A "calculable" idealized spiral flow system, consisting of a distributed aeration system resting on a partition wall with openings at the top and bottom has been studied. It is supported by cross walls making a "flow controlled aerating basin". The research work and full scale operation of activated sludge plants show that it is quite possible to aerate water to an extremely high oxygenation rate with a good economy by introducing air at low pressure and corresponding greater rates.

It seems possible to draw the following conclusions:

1.—At constant input of energy the oxygenation increases with decreased immersion depth of the air-distributor up to a well defined maximum at a certain depth, above which it decreases to be nill at the water surface. The most economical immersion depth of distributor is influenced by factors determining the pumping effect, such as distribution of air, head losses, and so on. It decreases with a good air distribution, and small water head losses, and the total effect increases simultaneously. It increases with increased losses in the aeration and water systems.

2.—At constant immersion depth of air distributor and constant aeration rate, the oxygenation increases rapidly with the number of air openings per unit area of distributor up to a certain density, above which only a slight increase occurs.

3.—At increased aeration level and about constant air flow per opening, the oxygenation increases directly or somewhat more rapidly than proportionally to the amount of air. This seems to be attributed to the increased air-water inter-face and turbulence. In pure water oxygenation rates above 1,200 ppm per hr are determined, and no upper limit has been found.

4.—At constant aeration level the oxygenation is practically independent of the basin depth.

5.—The economy seems to be about the same in a shallow (6 ft), a medium deep (10 ft to 15 ft) or a deep (20 ft) basin.

6.—The circulation rates are highly dependent on the shape of the basin and partition wall, the presence of deflectors and so on, in an irregular way at different aeration levels. Within wide limits it seems always possible however to obtain good circulation rates.

7.—This aeration principle can be adapted for use with different air distributors. The economy depends to a high degree upon the character and design of the distributor used.

8.—Aeration at low head is "self ventilating" and there is no risk of low oxygen or high carbon dioxyd content in the air above the basin in a covered plant.

The "rules" in point, 4, 5, and 6, which are valid between certain limits, will be helpful when making preliminary investigations or projects under different local conditions and assumptions of treatment.

This investigation indicates further possibilities in the development of low-head aeration of water and some additional research work is needed. It is however perhaps the first demonstration that it is possible to oxygenate water by medium bubble aeration with the same good economy as with fine bubble aeration or other good quality aerators, and with a capacity that far exceeds present need.

ACKNOWLEDGEMENTS

The author has co-operated with I. Gullström, F. ASCE, for several years on this subject and expresses his great gratitude to him. He also wishes to thank V. Jansa, F. ASCE and N. Westberg, for their valuable suggestions when preparing the paper. He is greatly indebted to Industrikemiska Aktiebolaget, Stockholm, for their generosity in defraying the expenses of the experimental and research work, and for their courtesy in publishing operating and test data, and further, for the great help given by the Director of the Company, Mr. Ohlander and the Research Engineers, Mr. Malm and Mr. Nordstroem. They have been granted patents in several countries for the design shown in Fig. 9.

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DISCUSSION

OPERATION OF A 7-MILE DIGESTED SLUDGE OUTFALL[a]

Closure by Norman B. Hume, Robert D. Bargman, Charles G. Gunnerson, and Charles E. Imel

NORMAN B. HUME,[1] F. ASCE, ROBERT D. BARGMAN,[2] F. ASCE, CHARLES G. GUNNERSON,[3] F. ASCE, AND CHARLES E. IMEL,[4] A.M. ASCE.—Messrs. Lawrance and Miller have commented upon the behavior of the floatables, apparent dilutions, sedimentation processes, and the cleaning of the sludge line. Observations made during the 2-yr period following that of the original investigation provide further information on these subjects.

Studies of various methods for removal of floatables have continued and it is expected that works for their removal will be in operation within a reasonable period. In this connection, waste activated sludge was pumped to sea and the receiving waters were closely studied. Microscopical examinations were made of water samples from various depths. It was found that the activated sludge floc, and particularly sphaerotilus, was elutriated from the discharge and distributed throughout the upper 100 ft while the field itself remained at depths generally greater than 200 ft. Accordingly, digestion is necessary for waste activated sludge which is to be discharged at sea.

Additional studies designed to evaluate initial dilution in the rising column have been made. It should be noted that only conservative properties such as temperature, salinity, or dissolved tracer concentration can be used for dilution computations. Non-conservative properties such as coliform density and suspended solids concentration provide information only upon the fate of the particular constituent. It has been found from detailed studies that as the discharge column rises through a temperature differential of about 4° F which usually obtains between the 200-ft level and the bottom, a minimum dilution of 75: 1 is effected.

Bottom samples taken over the 3-yr period of operation have shown that the maximum accumulation of sludge on the bottom has been held to less than 1 ft. This is the result of stabilization of the material by bottom organisms, intermittent winnowing by bottom currents, and periodic slumping of the deposits into the submarine canyon.[5]

One detailed survey of the receiving waters was made in order to characterize the deposition pattern of the sludge. Quantitative microscopical analyses

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[a] July, 1959, by Norman B. Hume, Robert D. Bargman, Charles G. Gunnerson, and Charles E. Imel.

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[5] "Characteristics and Effects of Hyperion Effluent in Santa Monica Bay, California," by N. B. Hume, C. G. Gunnerson, and C. E. Imel, paper presented at Annual Meeting, Pacific Sect., Amer. Geophysical Union, Univ. of Southern California, 1960.

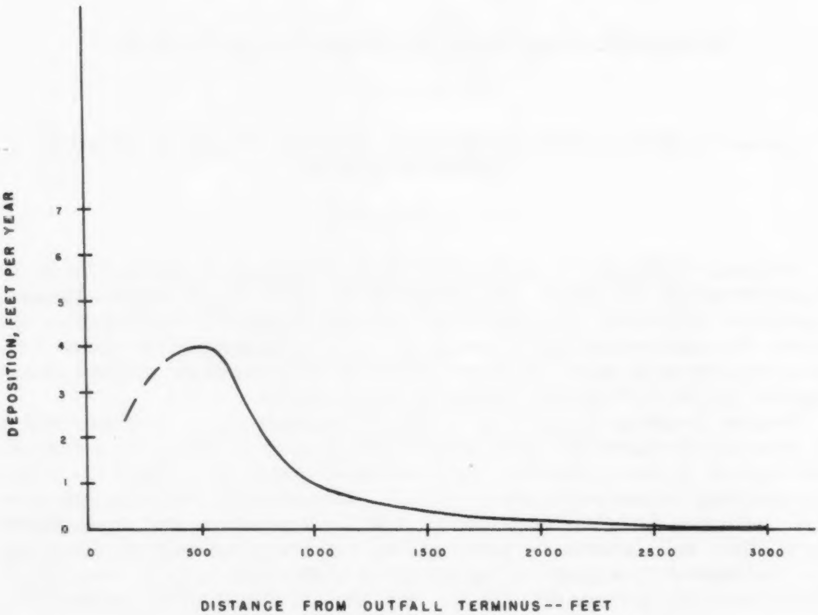

FIG. 1.—ESTIMATED RATES OF SLUDGE DEPOSITION AS DETERMINED FROM SUSPENDED SOLIDS CONCENTRATION IN WATER COLUMN

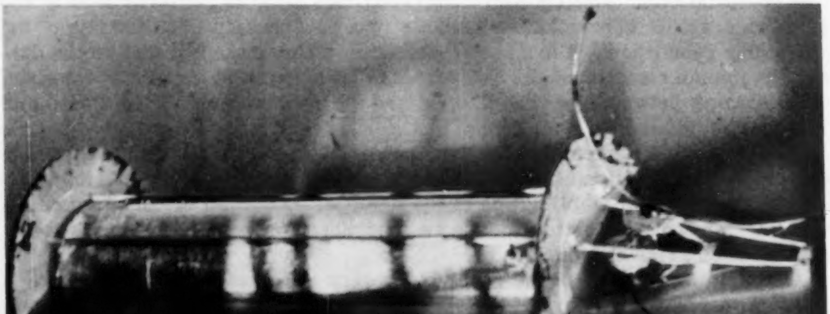

FIG. 2.—"PROVER" USED FOR SLUDGE LINE CLEANING. THE CIRCLES HAVE A DIAMETER OF ABOUT 20 IN.

involving staining of the entrained sludge with methylene blue were made of water samples from various depths throughout the receiving waters. The deposition rate of the sludge was estimated from the concentrations in the water. This indicated rate reached a maximum at a distance of about 500 ft from the outfall and became negligible at about ½ mile. These data are generalized in Fig. 1, which is based upon a bulk density of 10 pcf for the deposit.

Because of several variables in the observations and analytical techniques, the absolute values of the deposition rates are somewhat uncertain, although the relative values are probably good.

When viewed by underwater television, the superficial bottom deposits were found to vary between 1 in. and 6 in. in thickness and were in almost continuous motion from the swash of bottom waters.

Considerable physical and biological activity has been noted in the deposition area. Some 30% to 40% of bottom deposits were found to consist of worm

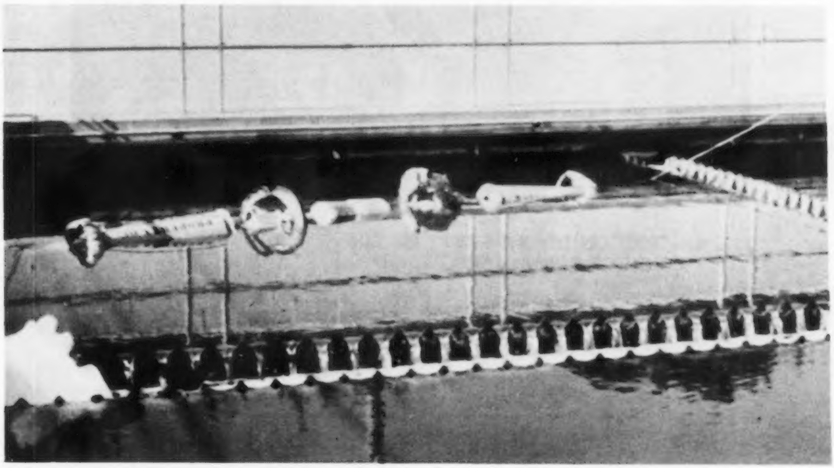

FIG. 3.—"PIG" USED FOR SLUDGE LINE CLEANING FLOATING IN FINAL TANK AT HYPERION

castings. The bottom deposits are not a firm material but are compacted to a bulk density of about 10 pcf. There has been no significant variation in the variety of species of fish and invertebrates in the area although some of the detritus feeders have shown large increases in population. Winnowing of the deposits is quite evident from the bottom sampling program which shows a periodic build up and removal of both total deposits and percent fines.

Other effects of bottom currents were found during a recent inspection of the pipe line in depths of from 60 ft to 145 ft. As noted by the authors, a series of anchors spaced at 500-ft intervals were installed to prevent lateral movement. Each anchor unit consists of two 4,500 lb concrete blocks connected by a 6-ft chain that lies across the pipe. A scoured area was found around each of the anchor blocks at all depths between 100 ft and 145 ft, the limit of the inspection. The scour is about 1 ft deep and extends about 10 ft in all directions

FIG. 4.—"PIG" ON DECK OF CITY OF LOS ANGELES OCEANOGRAPHIC SURVEY VESSEL, "PROWLER"

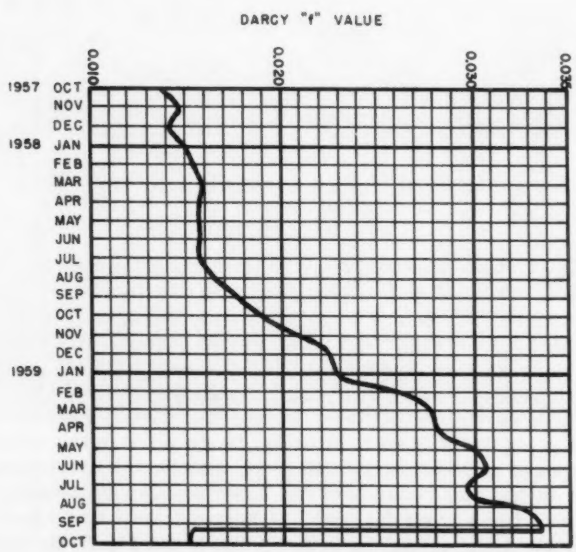

FIG. 5.—EFFECT OF SLUDGE LINE CLEANING UPON DARCY "f" VALUE

from the anchors. The pipe rests on the bottom in these areas. Normally it is bedded to just below the spring line. Along the pipe between the blocks there is sand deposition of from 2 in. to 8 in.

The sludge outfall cleaning, which was anticipated after 1 yr of operation, has been accomplished. A two-pass operation was used in the cleaning. A "prover" consisting essentially of a pair of 20-in. diam plywood circles (Fig. 2) was first sent through the line to insure that there would be no restriction that would prevent the passage of the articulated "pig" (Figs. 3 and 4), which included brush and scraper elements. Both the prover and pig passed through the line in about $3\frac{1}{2}$ hr, which is essentially the flow time of the diluted sludge. It is of interest to note that it was intended to recover the floating prover when it reached the ocean surface over the outfall terminus. Actually, currents moved the prover, which contained a sound signal that operated at 30-sec intervals, a sufficient distance during its ascent so that it was not seen. It subsequently washed up on the beach and was reported by bathers to be some sort of infernal machine. After appropriate examination by various ordinance experts, it was recovered by plant personnel.

Fig. 5 shows the return of the Darcy "f" to about its original value. This cleaning of the sludge outfall will be continued on a routine basis at intervals to be established—perhaps one or two year intervals.

ROLE OF PRICE IN THE ALLOCATION OF WATER RESOURCES[a]

Discussion by John A. Logan

JOHN A. LOGAN,[38] F. ASCE.—Mr. Hines has performed a valuable service to the civil engineering profession in presenting his analysis of the role of price in water resource allocation. This is a very much neglected area, particularly as regards municipal water supply, and engineers will welcome this important contribution to the literature.

The decision of the World Health Organization (WHO) and the International Cooperation Administration (ICA) to cooperate on a world-wide program of community water supply development is a case in point. The proposed program is based on the consideration of water as a saleable commodity and the water system as a utility. Both WHO and ICA recognize that, although water has an important public health significance, the cost of providing public supplies as a social benefit on a world-wide basis would be prohibitive. Water supplies must be promoted, therefore, on the basis of their financial self-sufficiency. This in itself is not enough. Because of the limited amounts of capital that are available in any given country, water supply has to compete for investment funds with such other factors as transportation, education, power, and irrigation on the basis of their contribution to economic development. Public water supply can no longer be taken for granted by engineers in developing areas for something inherently valuable which does not have to be justified on a cost-benefit basis.

During a conference (Washington, D. C., April 12-14, 1960) on the economics of public water supplies called by the Public Health Division of ICA, considerable attention was given to the role of municipal water supplies in industrial development and public health, and to an evaluation of the benefits in these areas. It was recognized, first of all, that many existing community water supplies are inadequate and do not meet the minimum requirements necessary for economic development. As minimum requirements, it was agreed that the supply; (1) be reliable; (2) provide 24-hr service; (3) provide adequate pressure; (4) be of good quality; (5) be adequate in quantity; and (6) be piped into or to the premises of all consumers, or made available nearby. It was also agreed that systems will attain their desired objectives only when effectively planned, operated, and managed.

A conference committee, consisting of Waldo, G. Bowman, F. ASCE, Wesley, E. Gilbertson, M. ASCE, Joseph Harantani, John A. Logan, F. ASCE, Luis Orihuela, Edmund G. Wagner and James D. Williams, considering the quantitative relationships between water supply and economic development, further agreed that most community water supply systems can be financed and built only to the extent that they attract investment capital. Greater emphasis must, therefore, be placed on the total economic advantages of municipal supplies

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[a] January, 1960, by Lawrence G. Hines.

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than has heretofore been the case. This will require that the public health as well as the industrial, commercial, and civic advantages of community water supplies be expressed in terms of economic benefits, insofar as this is possible.

The Committee expressed the following views:

Economic development in any given country begins when forces are mobilized to control and modify the physical environment in ways that yield productive change. This requires a change from the traditional agricultural economy (common to underdeveloped areas of the world) to an industrial economy, with increasing capital investments in service and industry. In the development from primitive to highly industrialized economies, the American economic historian W. W. Rostow has classified[39] five stages as follows: (1) traditional economies; (2) the transition period; (3) take-off; (4) the maturing economy; and (5) period of mass consumption. Each of these stages requires an increasing percentage of industrial workers and a larger capital investment in productive enterprise. Overhead capital investments are essential to provide the service facilities required, which include ports and docks, railroads, power, police protection, education, irrigation and, importantly, community water supplies.

Water is not only a physiological necessity but provides a service essential to public health and to commerical and industrial development. As well as being a key and essential element in this process, water supply systems can be self-supporting and self-generating. Water is a saleable commodity, and it is traditional in the United States and many other countries to pay for it by means of revenues from consumers and taxes from beneficiaries, which cover fixed costs, operation and maintenance. Wherever this custom has been established, the cost of water supply systems is being met by the communities served. Almost without exception electric light and power services, throughout the world, are operated on the basis of selling a commodity to the public which pays according to the amount used. A much larger number of people in the less-developed areas have installed electric services than water services. They also pay a much higher price for electricity. There is no inherent reason why community water supplies cannot be established on a similar basis.

Industrial and commercial development implies specialized labor and urbanization. Community water supply is essential to the development and maintenance of a vigorous, productive urban community. It is not merely a service but is, in effect, a spearhead of economic development.

While individual supplies can serve an interim need, experience has shown that these cannot meet the wide range of demands (residential, commercial, and industrial) which result from an expanding economy. If adequate, reliable supplies of water are not available, there is a definite limit to economic development. Provision must be made, not only for immediate needs, but for the expanding requirements which have been evidenced in countries such as the United States (approximately 50% of the U. S. municipal water supply in cities of over 10,000 is sold to commerce and industry). As William Mulholland stated in 1920, in answer to the query as to when Los Angeles would need the increased supply which he was advocating, "If you don't get it, you'll never need it."

The five essentials for industrial development in a community are labor supply, power, transportation, raw materials, and water supply. Water, like

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[39] "The Stages of Economic Growth," by W. W. Rostow, London, Cambridge Univ. Press, 1960.

power, fulfills not only a service function but has important collateral benefits. In providing water for commerce and industry, the economic base of the water supply system is extended; this permits economies in water production and lowers the cost for all consumers. Community water supplies not only stimulate industry in general but generate a series of specialized industries for the production of pipe, specials, bathroom and kitchen fixtures, and plumbing supplies, in general. New and improved housing is another important by-product resulting from community water supplies.

The availability of water is routinely used in the United States as an attraction to industry. The New York State Department of Commerce, for example, has a special report which is used to advertise the wide range of ground and surface supplies available for industrial use. The Tennessee Valley Authority (TVA) actively promotes industrial development in their area and encourages industry to locate where available supplies best meet their special needs. The Simpsonville-Fountain Inn area in South Carolina recently developed a new water supply for the primary purpose of attracting industry. A $900,000 bond issue was approved by a ratio of 64 to 1 by the population of 1,290. In two years, new industries alone provided employment for some 1,500 persons and property values have doubled and in some cases trebled.[40]

In Puerto Rico, the number of industrial customers served by the island-wide Water and Sewer Authority increased from 775 in 1949-50 to 1498 in 1958-59, an increase of approximately 200%. Industrial water revenues in 1958-59 amounted to $498,436.00, or about 7% of the total water revenue. The increase in commercial water sales during the same period was even greater and amounted to almost 30% of the 1958-59 revenue. Under the former unreliable municipal systems which existed before 1946, this phenomenal commerical and industrial growth would not have been possible.[41] Furthermore, commercial and industrial customers reduced the cost of water to residential customers.

Water supplies were built in key cities in the Rio Doce valley in Brazil about 1945, up to which time economic development had been very slow. The succeeding period, however, has been characterized by a rapid growth in industrial development and population, largely due to the availability of water.

A safe and adequate supply of water for human needs is absolutely essential to a healthy and productive life. One of the major causes of debility and illness in the less-developed countries are the diseases commonly transmitted through water or which are widespread due to lack of it. These include a variety of diarrheal and dysenteric diseases and a large group of virus diseases. Community water supply is not only a key element in the control of these diseases but the ready availability of water contributes substantially to the reduction of scabies, lice, and filth-borne diseases.

Several investigators have developed guides and approaches to the quantification of these public health benefits. Campbell and Morehead analysed[42] the population distribution in Brazil to determine the percentage in the productive age group, those between 20 and 65. The population in this group provides the wealth which supports the remainder, and to a large extent determines the

40 "South Carolina's Golden Strip Water District," A.W.W.A., Joint discussion, J.A.W.W.A., Vol. 48, November, 1956.

41 "Personal communication," by J. M. Henderson, February 15, 1960.

42 "Health as a Factor in Economic Development in Brazil," by E. A. Campbell and M. A. Morehead, Rio de Janeiro, Servico Especial De Saude Publica, Bulletin V, No. 2, 1952.

standard of living of the country as a whole. Weisbrod has developed[43] a stream of earnings method, based on the median income and consumption of high school graduates of the U. S. Weisbrod computed and plotted curves for the present values of net future earnings by age and sex using annual discount rates of 4% and 10%. Loss of life is considered as the loss of a productive unit. Wagner and Wannoni[44] in Venezuela analyzed the cost of community water supplies in terms of fixed costs, operation and maintenance. This cost was compared to the increase in productivity due to the reduction in premature death, illness and medical costs, as well as the savings effected in providing water by less efficient means. The return on the investment was calcuated to be 800%. Atkins has computed[45] the entire cost of water supplies and latrines in several countries as compared to the cost of typhoid fever, diarrhea and enteritis, and he reported that the entire cost would be recovered in from 2 yr to 5 yr.

Among other measurable returns from community water supplies is the increase in property values which inevitably occurs. This increase may be due to the convenience of having an adequate supply of pure water readily available, or because of the economic value of water to a business or industry occupying the property.

Another measurable return occurs when water carrying or vending systems are abandoned in favor of a piped supply. The manpower released from carrying water is thus made available for more productive purposes.

A return that is more intangible, but nevertheless very real, is the feedback which develops as a result of adequate community water supplies. Water triggers the upward spiral of improved health, social, and economic development. Better health permits more effective education which, in turn, stimulates increased productivity and a higher standard of living which compounds the upward trend. Concomittantly, the availability of water makes possible the development of commerce and industry which, because it also has this important feedback characteristic, provides further impetus to the upward spiral.

The report recommended that (1) methodology be developed for a more accurate determination of the cost-benefit ratios of community water supplies; (2) guidelines be prepared for the determination of the economic benefits of community water supplies; (3) that wide publicity be given to the ICA position with regard to community water supplies and economic development and that (4) ICA sponsor or co-sponsor a symposium on the economic impact of community water supplies in order to bring together engineers, economists, bankers, and others concerned with economic development and technical cooperation.

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43 "The Nature and Measurement of the Economic Benefits of Improvement in Public Health (with particular reference to cancer, tuberculosis, and poliomelitis)," by B. A. Weisbrod, Ph. D. Thesis, Northwestern Univ., June, 1958.

44 "Economias Previstas en Venzuela por Medio de la construccion de Abastos de Agua Potable en Zonas Rurales," by E. G. Wagner and L. Wannoni, Bulletin, Inter-Amer. Assn. of San. Engrg., Vol. 1, No. 4, 1948.

45 "Terminal Report Covering Services as Chief," by E. R. Jenney, Health and Sanitation Div., USOM/Brazil, August 1, 1959.

DIFFUSION IN A SECTIONALLY HOMOGENEOUS ESTUARY[a]

Discussion by M. B. McPherson

M. B. McPHERSON,[6] M. ASCE.—The author has contributed to a better understanding of the diffusion characteristics of the Delaware River estuary. His paper gives recognition to some of the estuarine problems common to oceanographers and civil engineers.

The theoretical presentation is quite extensive but somewhat indefinite with regard to "numerical application." Several important practical details have not been explained. Reference to the Delaware River is restricted to consideration only of the time and space distribution (flushing characteristics) of a conservative contaminant (dye) at High Water Slack (HWS) and at mean tide in the Waterways Experiment Station model at Vicksburg, Miss. The model—prototype salinity verification of Fig. 2 is presumed to apply to mean flow and mean tide at HWS. Instantaneous dye releases were employed in the model tests. Dye concentration was recorded at a single location at each of several model channel stations coincident with the occurrence of HWS at each sampling point. The method of sampling leads directly to one-dimensional or two-dimensional analyses, since it must be presumed that the dye concentration at the point of measurement is representative of that particular cross section at the instant of measurement ("uniform sectionally.") The fresh water inflow schedule for the Delaware River at Trenton, New Jersey (head of tide, at Sta. -160) and for major tributaries to the estuary was identical for Tests I and II. All components of the fresh water inflow schedule were maintained constant throughout the duration of the tests.

Some of the major reservations that must be considered before results of model tests can be applied to non-conservative wastes in the prototype for unsteady fresh water inflow conditions will be presented. The results of some recent model tests will be presented.

Movement of Dye.—

Peak Concentration versus Time.—The exponential peak concentration decay characteristic schematically illustrated in Fig. 3 is of considerable interest. However, plotting would indicate that only the values of Test II would yield an exponential relation similar to curve B for tidal cycles 5, 10, 15 and 20. Test I and Incodel Tests 10, 12, 13, and 14 (to be described) will satisfy an approximate, satisfactory exponential relation only beyond about 7 to 10 tidal cycles. This is not to say that the second term in Eq. 2 is not dominant after about three tidal cycles. By starting his numerical analysis at three cycles, the author has minimized the influence of the very steep initial decay rate, but whether or not a curve like B is approximated within the time of three or ten cycles has no bearing on his numerical analysis, even though such seems to be implied.

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[a] March, 1960, by Richard Kent.

[6] Prof. of Hydr. Engrg., Civ. Engrg. Dept., Univ. of Illinois, Urbana, Ill.

All four curves in Fig. 4 are for the same fresh water inflow schedule, given in Table 2.

The dye releases at Sta. +128 and Sta. +165 in Fig. 4 are the same as Tests I and II, respectively. However, dye corrections have been made to the raw data for Tests I and II but corresponding uncorrected data have been plotted in Fig. 4. With or without correction, however, there is a decided difference between the decay rates of peak concentration for various dye release locations.

The influence of fresh water inflow on the decay rate of peak dye concentration is illustrated in Fig. 6 for a two-dimensional laboratory flume. The high diffusive capacity of tidal action alone is clearly evident. It is also evident that higher fresh water inflow rates result in greater diffusion. Fig. 6 shows the effect of a 27-fold increase in fresh water inflow. A similar comparison for a four-fold increase in the Delaware model, for a dye release at Sta. 0+0, shows a minor difference between the curves (Incodel Tests 10 and 12). Data from another series of tests, for a three-fold and an eight-fold increase with dye release at Sta. +52, shows marked differences between the curves. It would appear that the location of dye release has a strong bearing on the comparative attenuation attributable to fresh water inflow, similarly as

TABLE 2.—TOTAL FRESH WATER INFLOW AT VARIOUS STATIONS; LONG-TERM MEANS

Approximate Station	R, Total, cfs
-160 (Trenton)	12,350
- 40	13,225
+ 60	16,475
+170	17,575
+235	18,025
+340	18,400
+420 (to Capes)	20,200

different release locations give rise to different decay rates for a given flow as shown in Fig. 4.

Time-Displacement of Dye.—The term S_p refers to the time of arrival at a given location of the peak concentration. The term M_p refers to the time of arrival at a given location of the centroid of the profile of the dye concentration along the axis of measurement. The term $\int \overline{v}\ dt$ represents the time-distance displacement of the fresh water inflow through the mean water prism. The "flushing anomaly" defined by Eqs. 3 and 4 can be partially explained. The profile of dye concentration is generally not symmetrical about the peak since the peak is skewed upstream, and S_p will lag the centroid, M_p.

The asymmetry is probably caused by increasing diffusion capacity downstream. If the profile of dye volume were taken instead of M_p, the skewness and resulting lag would be somewhat greater since the cross-sectional area continually increases from the head of tide to Miah Maull Shoal (Fig. 1). These remarks apply to both HWS and Low Water Slack (LWS). Unfortunately, the test data available to the author were confined to HWS concentration measurements. In Fig. 15 is given an approximate M_p curve for LWS obtained by interpolation of five Interstate Commission on the Delaware River (Incodel) flushing tests, made in 1959. The single asterisk in Fig. 15 indicates that the value of M_p for LWS is obtained by interpolation of Incodel tests. The double

asterisks indicates that the values, from Fig. 11, have been corrected for flow increment at sta. +170. In general, the mean displacement $\int \bar{v}\, dt$ will usually fall between the HWS M_p and LWS M_p. The author used a constant R of 16,475 cfs in computing the mean displacement in Figs. 11 and 12 rather than those given in the schedule of Table 15. The discrepancy in Fig. 11 has been corrected in Fig. 15, with the mean displacement curve tending more to the right beyond Sta. +170. A similar correction of Fig. 12 would make little difference until after 10 cycles, with the corrected curve 1.4 cycles below the author's at Sta. +230. The mean displacement would be closer to the LWS M_p than the HWS

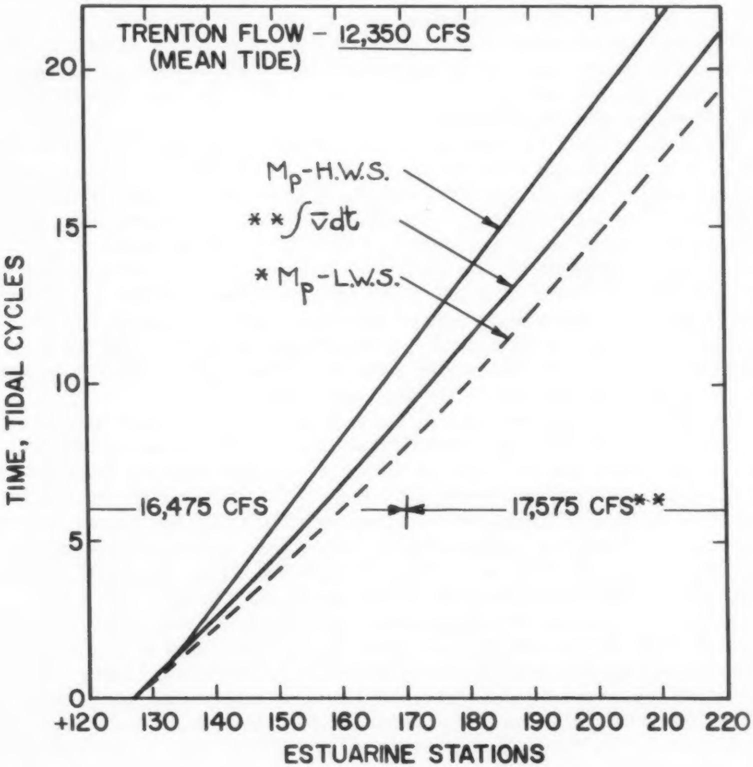

FIG. 15.—DELAWARE RIVER MODEL; TEST I - DYE RELEASE AT STA. + 127.

M_p. Cross-sectional areas given in the author's original report (28) (of which the present paper is a summary) were used in both instances.

The writer was concerned that storage effects might affect the mean displacement. Consequently, the figures presented by Wicker (29) for the same inflow schedule as Tests I and II, computed for the prototype by means of the "so-called cubage technique," were compared with the corrected mean displacement. There was a negligible difference between the corrected mean displacement and the storage balance figures. However, the areas used by the author and by Wicker may differ somewhat, obscuring any storage effect.

In Figs. 5, 11, and 12 and 15 through 19, M_p signifies the location of the centroid of the dye concentration profile. Since the volume per unit axial distance continually increases from the head of tide to near the mouth, a centroid of dye volume would be located further downstream than the centroid of dye concentration, M_p. For example, at ten cycles for Test II, the location of the centroid of the dye concentration profile is at Sta. +197.3 (see Fig. 12), but the centroid of the dye volume profile would be located at Sta. +201.6. Similar faster movement would generally be obtained for both HWS and LWS. Inasmuch as the dye concentration cannot be completely homogeneous across any section, use of a centroid of dye volume based on an axial profile is not necessarily better. However, the one-dimensional nature of the data makes any concomitant centroidal calculation an approximation. The dye corrections made by the author appear to have been applied as a constant percentage of the original data since the M_p and S_p for the corrected data are not noticeably different from those for the original data.

In Fig. 16 is shown a complete set of curves for Incodel Test 12 for dye release at Sta. 0+0, but with the same fresh water inflow schedule as used for Tests I and II. The values of S_p and M_p in Figs. 16, 17, and 18 are from the Delaware State Water Pollution Commission. These curves illustrate the arguments presented above in connection with the "flushing anomaly." The section areas given by the author (28) were for Sta. +100 to +370. It was therefore necessary to use other data for upstream stations and the mean tide, mid-stage areas given by Pritchard (9) were used to compute the mean displacement in Figs. 16 through 19. Downstream from Sta. +100 the mean displacement is a little further to the right in all four figures than it would be if the areas given by the author had been used instead.

Fig. 17 illustrates the greater movement times required with an R about one-fourth that for Fig. 16. Both figures are for dye release at Sta. 0+0.

Fig. 18 shows the influence of flow for releases near the head of tide and not far from the Trenton stream-flow gage. The dye concentration curves for these two tests are incomplete on the depletion side, but appear to be symmetrical and hence no differentiation between S_p and M_p is indicated in Fig. 18. Comparison of Test 13 in Fig. 18 with Fig. 17 for the same R indicates the general effect of the smaller section areas upstream.

In Fig. 19 are given the approximate, estimated movement curves for a release at head of tide but for the same R as Tests I and II, for comparison. These values are obtained by extrapolation of model flushing tests. It may be noted that the mean displacement would move faster than the LWS M_p up to around Sta. +30. For the lower flows of Fig. M-4, the mean displacement moves faster than the LWS M_p as far as about Sta. +100. The station to station variation between the mean displacement and M_p might be explained through the fact that the duration of ebb current exceeds that of the flood current by about 3 1/2 hr near the head of tide, by about 2 1/2 hr in the vicinity of Sta. 0+0 and by about 2 hr around Sta. +200. Proceeding downstream, as the duration of the ebb current decreases, the peak ebb currents tend to increase. One would therefore expect more intensive mixing between HWS and LWS as the tracer moves downstream.

"Numerical Application."—The author used the mean R salinity distribution at HWS (plotted in Fig. 2) in Eq. 54 to obtain K_s, the coefficient of "eddy diffusivity for salt." The "coefficient of eddy diffusion for pollutant" was taken as $K_s L_q/L_s$ (Eq. 65). The scales L_q and L_s are the ranges of pollutant and salt distributions. It is presumed that the range of salt was taken at about 330

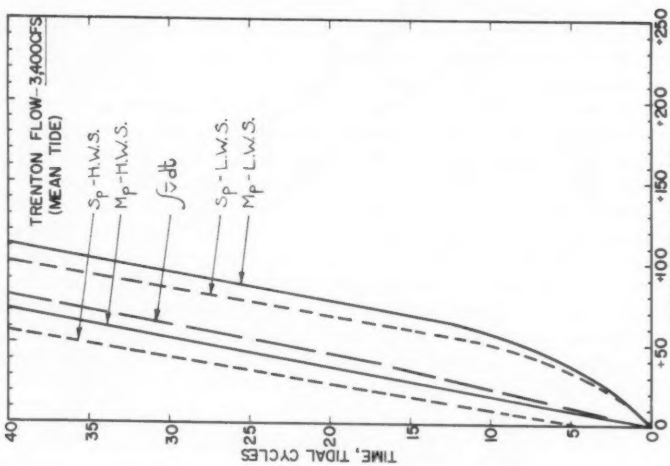

FIG. 17.—DELAWARE RIVER MODEL; INCODEL TEST 10 - DYE RELEASE AT STA. 0 + 0.

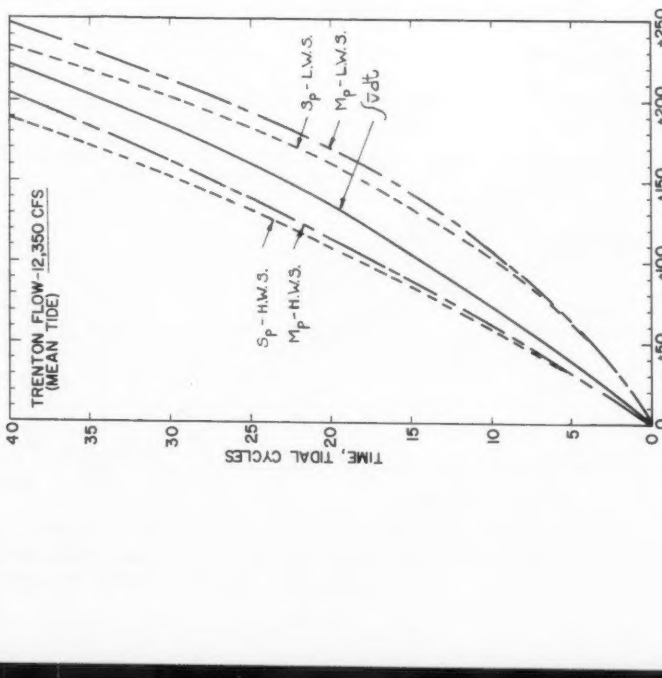

FIG. 16.—DELAWARE RIVER MODEL; INCODEL TEST 12 - DYE RELEASE AT STA. 0 + 0. FRESH WATER INFLOW SCHEDULE SAME AS TESTS I & II.

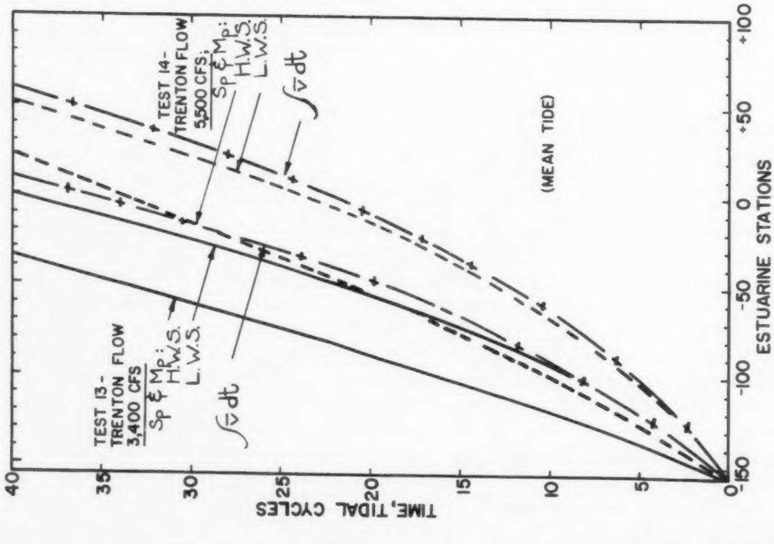

FIG. 18.—INCODEL TESTS 13 & 14 - DYE RELEASE AT STA. -155.

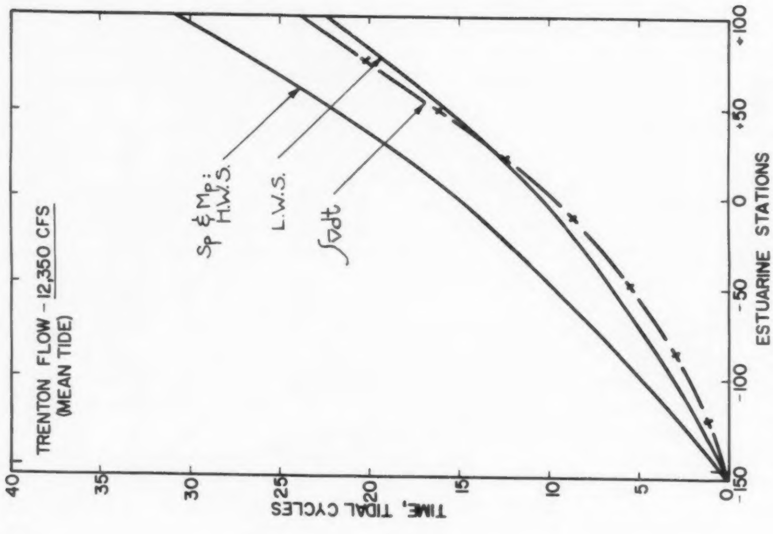

FIG. 19.—DELAWARE RIVER MODEL; FOR DYE RELEASE AT STA. - 155.

x 10^3 ft (prototype stations) and that the range of pollutant was in the order of figure given in Table 3.

The computed values of K_q were used in a solution of Eq. 37 (approximation of the classical molecular diffusion equation, reduced from a tensor to a vector form, Eq. 23). Computation via digital computer was started with the HWS dye concentration profile (corrected for dye loss) recorded for the model at the end of the third tidal cycle. It is stated that the boundary conditions given by Eqs. 33, 34, and 35 were approximately satisfied. Eq. 33 appears to be a general functional relationship describing the model dye concentration profile at three tidal cycles, the initial-value boundary condition. Continuity, or conservation of dye, is described by Eq. 35. (It is assumed that a numerical solution of Eq. 35 was performed separately on the corrected model data to check the dye loss allowances used). The results of the computer computations, and possibly some further computations by hand, are called "computed" in Figs. 11, 12, 13, and 14 and "predicted" in Figs. 9 and 10. These are compared with the model data, or with values calculated from the model data, and referred to as "observed." A recent annotated bibliography (30), in describing the author's report (28) says in part that "The theoretical predictions and the observed data for both tests show reasonable quantitative agreement." Rather, the results given in Figs. 13 and 14 show remarkable agreement if it is assumed that the

TABLE 3.—APPROXIMATE AXIAL DISTRIBUTION OF DYE[a]

Tidal Cycles:	5	10	15	20
Test I:	126	170	195	204
Test II:	133	189	196	223

[a] Concentration 0.005 ppm to 0.005 ppm, Feet (x10^3), Prototype Sta. (see Figs. 13 and 14).

dye corrections applied to the "observed" data resulted in a reasonable accounting for the total dye released.

The author performed another series of computations (28) with K_q constant (approximately equal to 3.5 x 10^{-3} sq ft per sec) in Eq. 43. The "computed" M_p and S_p for Test I fell above but near the mean displacement curve given in Fig. 11, falling far short of the "observed" in Fig. 11.

Values of K_q used by the author in Eq. 37 were not constant, but it appears that on the whole K_q is of the order of about one half of K_s. The writer computed the standard deviations of the distributions of salt and pollutant for Test II at ten cycles for which the ratio of the scales is about 0.4, as opposed to about 0.6 if Table 3 is reasonably correct. Hinze (31) suggests that Prandtl's theory leads to a ratio of momentum to scalar eddy diffusion coefficients equal to the square of ratio of the momentum to scalar mixing lengths. If the common assumption is made that these two mixing lengths are equal, then the eddy diffusion coefficients are equal and L_q must equal L_s for the author's K_q and K_s to be considered as eddy diffusion coefficients after Prandtl. However, the author's development of Eq. 65 does not necessarily imply that K_q is a momentum eddy diffusion coefficient.

The writer has been informed that a report concerning diffusion by homogeneous isotropic turbulence by J. C. Schoenfeld of the Netherlands Rukswaterstaat may soon be published. It is understood that this report sets forth a rigorous theoretical basis for analysis.

The author computed K_S with R constant at 16,475 cfs, whereas the stepped series of R given in Table 2 should have been used. As a consequence, the K_S values were all about 7% too low for stations beyond +170, but correction of this error would probably not affect the results.

Model Limitations.—"Extension of the flushing theory to the prototype estuary is presumed to be valid." Unfortunately, the only verification at hand is for salinity, which is excellent, as in the example used in Fig. 2. Flushing test verification is lacking. Hood (32) has described a number of tracers and concomitant detection apparatus, and Carpenter (33) reports on the use of Rhodamine B in dispersion tests in Baltimore Harbor.

Classification of verification tests will be complicated by the unsteady fresh water inflow in the prototype. Since the salt distribution is a complex function of R (readily solvable only by using the model as an analogue), the salt field is generally also unsteady. Further, unless a reliable steady-state tracer can be introduced or a suitable natural detector can be isolated above the salt field, the author's theory cannot be tested in the prototype in the important reach of the river that passes through the Greater Philadelphia Metropolitan Area.

TABLE 4.–PRODUCT OF FRESH WATER FLOW AT TRENTON IN cfs AND TIME IN TIDAL CYCLES FROM TRENTON TO STATION IN QUESTION[a]

Station	Mean Tide, Mean Displacement	M_p, HWS	M_p, LWS
- 80	42,000	82,000	46,000
- 38	78,000	135,000	84,000
+ 15	142,000	215,000	150,000
+ 50	197,000	270,000	200,000
+ 96	288,000	370,000	290,000
+126	345,000	445,000	345,000
+170	460,000	570,000	450,000

[a] From model interpolation and extrapolation; Steady flow.

If one assumes that the movement of fresh water inflow from Trenton is analogous to the movement of the centroid of dye concentration for steady flow from model tests, the generalized data in Table 4 will give some insight into the flow enigma. For a flow of 12,359 cfs at Trenton (Test I) at HWS, thirty-six cycles or about 19 days would be required for the flow to reach Sta. +126. To verify Test I, this flow at Trenton would have to be near-steady between about the nineteenth and about the thirtieth (twenty cycles earlier) day prior to the verification test. Presumably the salt field would be fairly steady if the above was approximated.

The model is a distorted scale type with horizontal scale of 1-to-1,000 and vertical scale of 1-to-100. The bulk Reynolds number of the prototype is 10^3 times the model. Roughness in the model has been simulated with localized roughness elements.

Kalinske (34) shows the variation of an eddy viscosity and diffusion coefficient with depth in the center of a smooth open channel, with the latter being a maximum at about 0.4 depth (from bottom), approaching zero at top and bottom. An extension of these experiments were reported by Kalinske and Robertson (35), again for marked immiscible particles. They conclude that "The turbulence near the surface of a stream is such that it has a much greater

ability to diffuse material laterally than vertically." Aside from the question of similarity, one wonders if the author's observation that "the spacial distribution of the dye rapidly attained a one-dimensional aspect," might more properly apply to the zone near the model water surface, for at least part of the time. In reading Laufer's paper (36), one notes that the microscales of turbulence, the intensities of turbulence and computed distributions of energy appear to be related to the mean transport velocity and channel size. However, any attempt to analyze idealized model diffusion characteristics is usually criticized on theoretical or physical grounds since the nature of diffusion in turbulent shear flows has not been defined. Discussion of the paper by Orlob (37) is an example.

Tests have been conducted on the Delaware River model using continuously released dye in an effort to achieve an analogue for estimating asymptotic ultimate contaminant concentrations. Aside from the basic objections to the validity of such an analogue, procedures for dye loss corrections are necessarily arbitrary and no doubt quite inexact. The author has presumably made data corrections consistent with the conservation of the total released dye. It would seem more appropriate, therefore, to integrate the model results for a balanced, instantaneous dye release. Notwithstanding the possibility of achieving a reliable continuous dye release profile, prototype application requires a steady fresh water inflow schedule of exceedingly long duration. One of the objections to the use of continuous dye release results from the model for evaluating the effects of non-conservative wastes arises from the fact that convective mixing is involved in an important part of the oxygen balance while the diffusion of oxygen across the water-air interface is usually regarded as predominantly a molecular type. While the eddy diffusivity, or turbulent diffusion, might be conceived as the product of the intensity and scale of the turbulent mixing process (another way of arriving at Eq. 65), a realistic allocation of the contributions from molecular and turbulent diffusion would necessarily have to be appraised using discrete measurements made within both model and prototype systems (11). It is commonly assumed that an analogy exists between molecular and turbulent diffusion, as between Eqs. 23 and 65. The author's results imply that such an analogy does exist in the instantaneous dispersion of a conservative contaminant in the model. Any literal attempt to stretch this limited analogy to the dispersion of a non-conservative contaminant in the prototype must be considered highly suspect.

An interesting analysis of continuous dye release tests in a laboratory flume has been presented by Huiswaard, Banks, and Bell (39). In addition, they reviewed the author's work and proposed a spacially dependent pollutant diffusion factor, which they found to be about one fourth of K_s, but did not have the opportunity of performing a solution of Eq. 37 for a comparison with the model test results. As with the author's procedure, their suggested approach would test only the suitability of their semi-empirical relations in regenerating concentration profiles from components of the model test profiles, with which the predicted results would later be compared.

Non-Conservative Pollution Studies.—As mentioned previously, one approach currently being used to evaluate non-conservative wastes in the Delaware estuary presumes a direct analogy between prototype waste distribution and the distribution of dye observed in continuous release model tests. A procedure for extrapolating estimated Delaware estuary waste loadings for prescribed dissolved oxygen levels has been advanced by Niles (40).

Recently, a regression analysis was performed on temperature and dissolved oxygen data accumulated over a period of 9 yr, under the assumption of

a sinusoidal variation. Three time-components were utilized. The general features of this approach have been reported by Diachishin (41).

The latter two studies utilized the appropriate data from monthly boat samples collected jointly by the U.S.G.S. and the City of Philadelphia.

After the author's paper was published, O'Connor (42) compared computed with observed dissolved oxygen profiles for two HWS and one LWS boat sampling run made in the latter half of 1956, by the state of Delaware's Water Pollution Control Commission. The profiles used were for the rising end of the curves beyond the minimum concentrations, for five sampling points extending from Sta. +193 to Sta. +305. Since the data were collected at the "same slack" and the river flow was "low," it was assumed that conditions were steady and that diffusion coefficients used were calculated from the observed chloride profiles. In addition, it was assumed that the diffusion coefficient, the channel cross section and the mean transport velocity were constant over the reach studied. O'Connor suggests that the distribution of a stable dissolved chemical ion might be utilized as a spacial tracer for the section of the river upstream from the salinity front. Since there are no significantly distributed spacial tracers extant in most of the nonsaline section, a special tracer would have to be introduced prior to sampling. Although O'Connor cites fresh water inflow rates, they are not necessarily the true rates which contributed to mixing and transport.

In summary, it may be fairly stated that the several attempts to establish reasonably exact relationships between waste loadings, temperature, dissolved oxygen and inflow have fallen far short of the mark. Analyses based upon the data from occasional boat sampling runs, whether taken monthly throughout the year or at the same slack during the period of low flows, appear to be inadequate. Automatic, continuous dissolved oxygen and temperature recorders are to be installed in the estuary. Through a cooperative agreement the City of Philadelphia and the U.S.G.S. expect to have three units in operation in the summer of 1960, with several more planned for the near future. The recorder data will be subjected to a power spectrum analysis (43).

It is evident that flushing data contribute to an understanding of some of the factors affecting waste distribution in an estuary. If by the statement "In particular, a means of predicting the quantities of public and industrial wastes that can be removed and neutralized by natural circulatory systems is desirable," the author is implying that the means will arise from solutions to flushing problems, he has vastly oversimplified a complex situation. Estuarine assimilation of non-conservative wastes is the least understood liquid-waste disposal problem. The continued close coordination of efforts between engineers and oceanographers is needed.

Acknowledgments.—Data for the five Incodel flushing tests were made available to the writer by the Philadelphia Water Department. The Department supported Incodel Tests 13 and 14. Some of the material presented in this discussion had been prepared for the Department while the writer was serving in a consulting capacity. Acknowledgment is hereby made to Commissioner S. S. Baxter for permission to use the material.

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38. "Advanced Fluid Mechanics," edited by H. Rouse, John Wiley, Inc., New York, 1959, IV-D, 444 pp.

39. "Dispersion in Tidal Estuaries," by P. J. Huiswaard, R. B. Banks, and C. G. Bell, Jr., Report to AEC, AT(11-1)-353, Tech. Inst., Northwestern Univ., Jan. 1, 1959, 99 pp.

40. "Dispersal of Pollution by Tidal Movements," by T. M. Niles, Proceedings ASCE, Vol. II No. SA5, October, 1957; Discussions, Vol. 84, No. SA2, April, 1958, Vol. 84, No. SA3, June, 1958.

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43. "The Measurement of Power Spectra," by R. B. Blackman and J. W. Tukey, Dover Publications, New York, 1959, 190 pp.

FUNDAMENTAL CONSIDERATIONS IN HIGH RATE DIGESTION[a]

Discussion by F. Sulzer, C. E. Keefer

F. SULZER.[16]—The authors suggest that the rate of digestion can be theoretically explained by applying the laws of chemical reactions to the equation

$$\text{"organic matter + bacteria} \longrightarrow CH_4 + CO_2\text{"}.$$

Noting that the equation corresponds formally to a binary (that is, bimolecular?) reaction and that the "bacteria" remains essentially at constant concentration, the authors conclude that ". . . the kinetics are the same as those for unimolecular reactions . . . . ."

Such a formulation suggests that complex biological systems might be analyzed in a simple fashion by arranging any type of reactants in an expression analogous to chemical reaction equations and by applying, consequently, the kinetic laws for chemical reactions in the homogenous phase to these "overall" reaction expressions. Such an explanation for the observed first-order reaction rate is, however, rather misleading. (The expression "first order reaction" is used in preference to the terms "unimolecular" or "monomolecular reaction" if the reaction mechanism is not definitely known to exclusively involve one molecular species.) A fundamental understanding of the removal kinetics can only be gained from a study of simpler systems than sludge digestion.

The reduction of volatile solids in sludge digestion can, in principle, be considered as a biological process of substrate removal (substrate utilization, respectively) by microorganisms. Microbiologists working with isolated cultures have been using so-called "washed cell suspensions" for a long time as simple systems to study substrate utilizations. With washed cell suspensions the metabolic processes involving specific compounds can be studied without interference from unknown substrate compounds and without the effects of growth which frequently complicate and obscure the kinetics of a substrate-microorganism interaction. In the following discussion it is tacitly assumed that, in the systems considered, the increase in active cell material during the substrate removal is negligible in comparison to the amount of active cell material present.

In most studies with washed cell suspensions, substrate utilization has been followed by indirect means (such as carbon dioxide evolution or, with aerobic systems, oxygen uptake). Data on such indirect measurements of removal rates are profuse and widely scattered throughout the biological literature.

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[a] March, 1960, by C. N. Sawyer and J. S. Grumbling.

[16] Asst. Prof., Dept. of San. Engrg., Univ. of North Carolina, Chapel Hill, N. C.

They, however, furnish only indirect evidence of the substrate removal process. In some instances, the substrate has been determined directly.[17] The experiments show that, usually, the rate of removal of simple, specific compounds is constant, in other words, the reaction rate is zero-order. In some cases, diminishing rates of substrate removal, depending on organism and substrate, have been observed, but this can be expected since interference by inhibitory metabolic products is possible. Enzyme adaptation mechanisms also may alter the rate. However, one is led to the conclusion that the removal of simple, specific compounds is, in principle, a zero-order reaction. This is in

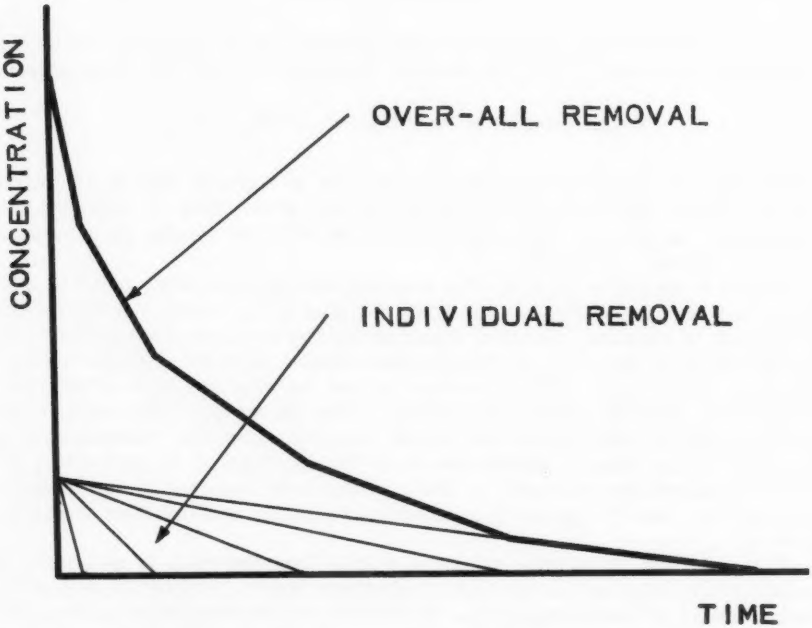

FIG. 10.—COMPONENT SUBSTRATE REMOVAL

agreement with the laws governing enzymatic reactions (if the apparent Michaelis constant of the rate determining enzyme is assumed to be very small).

Zero-order removal rates have not only been demonstrated under the restricted conditions mentioned above, but also, for example, in industrial fermentation processes.[18,19] Mixed cultures also give constant removal rates

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17 "The Adaptability of Glucozymase and Galactozymase in Bacterium Coli," by M. Stephenson and E. F. Gale, Biochemical Journal, 1937, Vol. 31, pp. 1311-1315.

18 "The Effect of the Carbohydrate Nutrition of Penicillin Production by Penicillium Chrysogenum Q-176," by F. V. Soltero and M. J. Johnson, Applied Microbidogy, 1953, Vol. 1, pp. 52-57.

19 "Itaconic Acid by Fermentation with Aspergillus Terreus," by V. F. Pfeifer, C. Vojnovich, and E. N. Heger, Industrial Engrg. Chem., 1952, Vol. 44, 2975-2980.

for simple substrates. Wuhrmann et. al.[20] showed that a variety of monomer compounds (carbohydrate, fatty acid, amino acid) are removed at constant rates by activated sludge. Data presented[21] by Weston and Stack indicate a zero-order BOD removal for wastes containing preponderantly sucrose (The authors explain this as the resultant of increasing metabolism due to growth and concurrent release of metabolic products by cell lysis, but there is hardly sufficient evidence for such an interpretation.). Similar evidence comes from laboratory soil percolation data,[22] which show that ammonium ion is oxidized to nitrate by an established soil microbe population at constant rate.

How then is the observed first-order reaction rate in sludge digestion to be explained? The same question can as well be posed for the aerobic processes of waste removal (trickling filter, activated sludge), where a similar first-order rate is observed. First, it should be recognized that the parameters used in measuring waste removal rates are unspecific (BOD, volatile solids, or similar units) and, therefore, express some over-all effect. Secondly, there are innumerably many compounds present in an ordinary sewage-type waste. If we assume that each compound is removed at a zero-order rate specific for the compound (and the microbial culture) and that these specific removal rates differ considerably, the over-all effect will approximate a first-order reaction. A simple model (Fig. 10) should illustrate the concept. Assume a substrate consisting of five different chemical compounds, present in equal concentrations (determined in some unspecific measure like BOD). As the individual removal rates vary widely, the over-all effect will simulate a first-order reaction or similar diminishing rate reaction (Fig. 10).

The first-order reaction rate in sludge digestion and similar biological processes can therefore be explained as the sum of a large number of zero-order reaction rates. It is realized that this concept needs further investigation and broader experimental proof, since many assumptions were made and the role of high-molecular substrates and suspended solids has not been dealt with.

C. E. KEEFER,[23] F. ASCE.—In his discussion of high-rate sludge digestion Clair N. Sawyer rightly indicates that the mixing of the contents of digesters becomes a difficult problem, especially when the concentration of the solids approaches 6%. When raw sludge is introduced into a digester containing digested material and the two materials are mixed together, it is not only difficult to mix intimately and thoroughly the two sludges, but when sludge is withdrawn day by day equal in amount to the raw sludge added, the sludge that is removed will contain considerable quantities of partially digested solids. This situation can be illustrated by the following example. Let it be assumed that a digester is filled with 100 units of digested sludge and that under ideal conditions of temperature, proper mixing, and so forth, raw sludge will digest in 10 days.

To simplify the computations let it be assumed that the percentage of organic matter digested day by day is directly proportional to the period of digestion,

20 "On the Theory of the Activated Sludge Process," by K. Wuhrmann and co-workers, Schweiz, Zeits. Hydrol., 1958, Vol. 20, pp. 284-310, 311-330.

21 "Prediction of the Performance of Completely-Mixed Continuous Biological Systems from Batch Data," by R. F. Weston and V. T. Stack, Conf. on Biol. Waste Treatment, Manhattan College, No. 24, 1960.

22 "Biochemistry of Nitrification in Soil," by J. H. Quastel and P. G. Scholefield, Bact. Revs., 1951, Vol. 15, pp. 1-53.

23 Sewage Engr., Dept. of Pub. Wks., Baltimore, Md.

that is 50% of the organic matter is digested in 5 days and 100% in 10 days. Although this assumption is, strictly speaking, not correct, it does not differ from the actual facts so as to greatly change the results. Let it be assumed that at the beginning of each day 10 units of sludge are removed from the digester, 10 units of way sludge are then immediately added to the digester and the tank contents are thoroughly mixed. The following tabulation gives the percentage of partially digested solids in the sludge withdrawn day by day up to the end of 20 days.

Digestion Time, days	Percentage of partially digested sludge in the material withdrawn
1	9.0
2	16.3
3	22.2
4	27.0
5	30.9
6	34.0
7	36.5
8	38.6
9	40.3
10	41.6
15	45.4
20	46.7

These figures indicate that at the end of 10 and 20 days the sludge withdrawn will contain 41.6% and 46.7% of partially digested solids, respectively. Although these percentages are admittedly high because the digestion of sewage solids is more nearly a unimolecular reaction than one where the reaction rate is constant, the computations indicate that if daily additions of raw sludge are added to a digester filled with ripe sludge and the tank contents are then thoroughly mixed and sludge withdrawals are made equal in amount to the sludge additions, the sludge withdrawn will contain appreciable amounts of sludge that has not been thoroughly digested.

The foregoing difficulties associated with high-rate sludge digestion have been avoided at the Patapsco and the Back River sewage treatment plants in Baltimore, Md. by pumping the raw sludge into a small premixing tank, into which ripe sludge is drawn from the digester. These two sludges are thoroughly mixed for about 5 min, and the mixture is then introduced into the digester. Since each daily addition of raw sludge is pumped into the top of the digester and since the contents of the digester are not stirred, the rate at which the raw sludge moves downward to the outlet pipe depends on the amount of digested sludge that is removed and disposed of daily from the digester. The experiments that were conducted in Baltimore[24] using a digester with a working capacity of 180,000 cu ft indicated that satisfactory digestion could be obtained in about 10 days. It was found essential to thoroughly premix the raw and the digested sludges before introducing them into the digester so that each particle of raw material was brought into intimate contact with an ample quantity of digested solids. The best results were obtained when about one part, by weight, of raw volatile solids was mixed with one part of digested volatile solids.

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24 "Effects of Premixing Raw and Digested Sludge on High-Rate Digestion," by C. E. Keefer, Sewage and Industrial Wastes, Vol. 31, No. 4, April, 1959, p. 388.

In view of the foregoing discussion, it is considered preferable to seed the raw sludge by premixing it with an adequate amount of digested sludge for several minutes in a small tank outside of the digester and then introducing the mixture into the digester. By following this procedure the raw solids can be thoroughly mixed with ripe sludge and there is no danger of drawing off sludge that has been in the digester for only a short period of time.

DIFFUSERS FOR DISPOSAL OF SEWAGE IN SEA WATER[a]

Discussion by J. M. Jordaan, Jr., C. H. Lawrance

J. M. JORDAAN JR.[20]—For the sake of academic interest the writer would like to ask whether a deflector on each diffusor outlet, so arranged as to eject the jets downward against the ocean floor, would not effect immediate and more effective mixing of sewage effluents with the colder bottom layers of the sea water. The upward rising plume is described by the authors as dipping down again at the surface, presumably because it is heavier than the surface layers, although its residual momentum first carries the effluent plume to the surface. Would it not therefore be possible to effect the mixing almost entirely by jet action so that the plume, mixed with colder, heavier water will tend to stay down?

Directing the jets vertically down will no doubt give rise to scour but they could be directed at, say, 45° down, against a protective slab covering the impact area. The jet would strike the bottom and spread out similarly to the way in which a downward directed jet of steam and vapour from a steam pipe, after striking the floor, wells up in a highly turbulent roller. A somewhat higher manifold pressure and smaller ports would be required to produce a high efflux velocity, leading to better manifold flow distribution as well. The toroidal vortex system would rise much slower around each outlet nozzle after striking the bottom and due to the central "hole" would have a greater contact zone over which turbulent mixing can occur. The effluent will also stay in contact with the bottom longer where the orbital motion of the long period surface waves sets up a turbulent zone of a few feet in depth increasing the mixing opportunities. It seems quite possible to create a submerged sewage field as also postulated by the authors. If dilutions of 10^3 times or better could be obtained by intense jetting against pads on the ocean floor, there may be no appearance of sewage effluents above the thermocline or on the sea surface at all.

C. H. LAWRANCE; M. ASCE,[22]—This paper has been particularly interesting to the wrtier, who participated in the investigations and design of the 5-mile long Ocean Outfall for Effluent Disposal for the City of Los Angeles Hyperion Treatment Plant. During the course of the design of the diffuser for the Hyperion Outfall, the engineers made use of the principles and methods presented by the authors. The authors' paper condenses a great deal of information and findings resulting from the operations of the outfalls at Whites Point, yet the presentation ably presents the most important findings and conclusions.

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[a] March, 1960, by A. M. Rawn, F. R. Bowerman and Norman H. Brooks.

[20] Head, Hydr. Sect., Natl. Mech. Engrg. Research Inst., S. A. Council For Scientific and Industrial Research, P. O. Box 395, Pretoria, South Africa.

[22] Proj. Engr., Koebig and Koebig Cons. Engrg. Architecture, Los Angeles, formerly San. Engr., Hyperion Engrs., Los Angeles, Calif.

The writer wishes to underscore the statement made by the authors' that the advantage of higher initial dilutions achieved with diffusers is partially lost because the dilution rate occurring in dilute sewage fields formed by diffusers is slower than that occurring in the narrow field originating from an open pipe. This is quite evident upon examination of the mathematical formulations of Rawn and Palmer[23] and of Brooks,[24] and is also borne out convincingly by field observations. It applies both to the matter of physical dilution, as may be computed from chemical and physical measurements, and to "apparent" dilution, as may be measured by bacterial densities.

A case in point is the experience of the Los Angeles County Sanitation Districts themselves upon completion of the first two diffusers for the 60-in. and the 72-in. outfalls, respectively. During the course of the rather extensive investigations undertaken by the Engineers incidental to the design of the Hyperion 5-mile outfall, the writer had occasion to analyze the laboratory data from a regular State Department of Public Health sampling, at the beach sampling stations, in the general vicinity of the Whites Point outfalls, for a control period before the construction of the diffusers, and for a similar period following completion of construction of these diffusers. For the period of January 7, 1952, through January 27, 1953, most probable numbers of coliform organisms at individual sampling stations were compiled and geometric means computed. Similarly for the period April 1, 1954, through December 20, 1954, representing the first period of use of the new diffusers, beach sampling station coliform densities were also compiled and geometric means computed.

Now the provision of diffusers on these two outfalls had resulted in a substantial improvement in initial dilution of the plant effluent upon submarine injection. The authors' Table 2 indicates that about 7.4 to 7.5 times as much initial dilution, average for the two outfalls, was obtained by the use of diffusers as was obtained from the open outfall pipes prior to these diffusers. It is noteworthy that the coliform densities at the shore stations did not decrease with corresponding magnitude. The decrease ranged from 1.40 times at the most distant station (Station 0, 24,000 ft oblique distance from the outlets) to 5.15 times at Station 3, the origin of the ocean outfalls. The average decrease for all stations studied was by a factor of 2.63. This compares with the increase in initial dilution of about 7.4.

The new 5-mile Hyperion Ocean Outfall for Effluent Disposal terminates in a large diffuser in nearly 200 ft depth of water and discharges an unchlorinated mixture of primary effluent and standard rate activated sludge effluent into Santa Monica Bay. The outfall and diffuser were completed in the early part of 1960, and have been in general use since that time. It is gratifying to find that their performance has so far come up to expectations regarding initial dilution, subsurface stratification of the field, and coliform densities, both offshore and at the beach stations. Piezometer connections were incorporated into the design at the halfway point in the outfall, at the beginning of the diffuser, at the point of pipe size reduction in each diffuser leg and at the terminus of each diffuser leg. Initial measurements of pressure made at these

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23 "Predetermining the Extent of a Sewage Field in Sea Water," by A. M. Rawn and H. K. Palmer, Transactions, ASCE, Vol. 94, 1930, pp. 1036-1081.

24 Brooks, N. H., "Diffusion of Sewage Effluent in an Ocean Current," Proc. First International Conference on Waste Disposal in the Marine Environment, Univ. of California, July, 1959. The basic formulations are also presented in Dr. Brooks' Appendix A, "Methods of Analysis of the Performance of Ocean Outfall Diffusers with Application to the Proposed Hyperion Outfall," to accompany Hyperion Engineers' "Report Relating to Length of Outfall and Types of Diffusers for the Ocean Outfall for Effluent Disposal, April 6, 1956."

points from surface vessels and from the stationary laying barge prior to acceptance of the construction job indicated general agreement with anticipated hydraulic conditions. Additional data will be obtained from time to time in the future by the City of Los Angeles Bureau of Sanitation. These data, particularly those obtained after flows are substantially increased and piezometric pressures are higher, will be of value in establishing the hydraulic performance of the diffuser.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

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SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158 (IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176 (WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184 (WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205 (PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214 (SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223 (ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240 (PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248 (SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278 (HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287 (PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296 (SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322 (ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330 (CP2).

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JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411 (SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436 (ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445 (EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454 (EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA3), 2475(ST5), 2476(SA3), 2477(ST5), 2478 (HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487 (CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502 (EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511 (EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520 (PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529 (HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

JULY: 2541(ST7), 2542(ST7), 2543(SA4), 2544(ST7), 2545(ST7), 2546(HY7), 2547(ST7), 2548(SU2), 2549(SA4), 2550(SU2), 2551(HY7), 2552(ST7), 2553(SU2), 2554(SA4), 2555(ST7), 2556(SA4), 2557(SA4), 2558(SA4), 2559(ST7), 2560(SU2)[c], 2561(SA4)[c], 2562(HY7)[c], 2563(ST7)[c].

AUGUST: 2564(SM4), 2565(EM4), 2566(ST8), 2567(EM4), 2568(PO4), 2569(PO4), 2570(HY8), 2571(EM4), 2572(EM4), 2573(EM4), 2574(SM4), 2575(EM4), 2576(EM4), 2577(HY8), 2578(EM4), 2579(PO4), 2580 (EM4), 2581(ST8), 2582(ST8), 2583(EM4)[c], 2584(PO4)[c], 2585(ST8)[c], 2586(SM4)[c], 2587(HY8)[c].

SEPTEMBER: 2588(IR3), 2589(IR3), 2590(WW3), 2591(IR3), 2592(HW3), 2593(IR3), 2594(IR3), 2595(IR3), 2596 (HW3), 2597(WW3), 2598(IR3), 2599(WW3), 2600(WW3), 2601(WW3), 2602(WW3), 2603(WW3), 2604(HW3), 2605(SA5), 2606(WW3), 2607(SA5), 2608(ST9), 2609(SA5)[c], 2610(IR3), 2611(WW3)[c], 2612(ST9)[c], 2613(IR3)[c], 2614(HW3)[c].

c. Discussion of several papers, grouped by divisions.

PART 2

SEPTEMBER 1960 — 32 NO. SA 5
VOLUME 86 PART 2

Your attention is invited

NEWS
OF THE
SANITARY ENGINEERING DIVISION
OF
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AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

JOURNAL OF THE SANITARY ENGINEERING DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

September, 1960

NEW VISTAS IN SANITARY ENGINEERING

In past issues of the Newsletter, reports have been made of interstate enforcement actions taken by the Department of Health, Education, and Welfare under terms of the Federal Water Pollution Control Act. These actions have resulted in the development of new concepts in the field of administrative law, and continuing cases are building a background in this area. Generally, these new concepts are concerned with actions of one level of government—in this case Federal—against another level of government—primarily municipal.

While news media and professional journals have reported action taken in most enforcement cases, the text of orders and findings have not been published for wide distribution. So that sanitary engineers may read one of these in full text, we reproduced below the Notice of the Secretary of Health, Education, and Welfare and Findings, Conclusions and Recommendations of the Hearing Board in connection with pollution of the interstate waters of the Missouri River in the Kansas Cities Metropolitan Area.

Note.—No. 1960-32 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. SA 5, September, 1960.

THE SECRETARY OF HEALTH, EDUCATION, AND WELFARE
WASHINGTON

SEAL

In the Matter of the
POLLUTION OF THE INTERSTATE WATERS
Of The
MISSOURI RIVER AND CONNECTING OR TRIBUTARY WATERS IN OR ADJACENT TO THE KANSAS CITIES METROPOLITAN AREA

NOTICE

There is attached hereto, and made a part hereof, the Findings, Conclusions, and Recommendations, dated June 17, 1960, of the Hearing Board, convened pursuant to the provisions of section 8(e) of the Federal Water Pollution Control Act [33 U.S.C. 466g (e)], which held a public hearing in the matter of pollution of the interstate waters of the Missouri River and other interstate waters connecting therewith or tributary thereto in or adjacent to the Metropolitan area of Kansas City, Kansas; Kansas City, Missouri; North Kansas City, Missouri.

The Board found that the cities of Kansas City, Kansas; Kansas City, Missouri; North Kansas City, Missouri; and the firms named below are discharging sewage, industrial, and other wastes causing or contributing to pollution of interstate waters which endangers the health or welfare of persons in a State other than that in which the discharges originate, by discharges into or reaching the Kansas River; Turkey Creek Sewer or the Missouri River.

In accordance with section 8 of the Federal Water Pollution Control Act, the City of Kansas City, Kansas; the City of Kansas City, Missouri; the City of North Kansas City, Missouri; Fairfax Drainage District of Kansas; Kaw Valley Drainage District of Kansas; and the following named industrial establishments:

Kansas City Stockyards Company
Livestock Exchange Building
1600 Genessee Street
Kansas City, Missouri

Bio-Laboratories, Inc.
50 North 2nd Street
Kansas City, Kansas

National Cylinder Gas Company
1614 State Avenue
Kansas City, Kansas

The National Laboratories Corporation
1721 Baltimore Avenue
Kansas City, Missouri

Phillips Petroleum Company
2029 Fairfax Trafficway
Kansas City, Kansas

Proctor & Gamble Manufacturing Company
1900 Kansas Avenue
Kansas City, Kansas

Atchison, Topeka, and Santa Fe
Railroad Company
Fairfax Building
Kansas City, Missouri

Sinclair Refining Company
906 Grand Avenue
Kansas City, Missouri

Swift & Company
10 Berger Avenue
Kansas City, Kansas

Midwest Cold Storage & Ice Corporation
1101 South 5th Street
Kansas City, Kansas

Buick, Oldsmobile, & Pontiac
Assembly Plant
100 Kindleberger Road
Kansas City, Kansas

are hereby notified and directed to cease and desist from discharging, without proper, adequate and effective treatment, such sewage, industrial, and other wastes into said waters and to secure abatement of such pollution, by the completion and placing into operation of proper, adequate, and effective municipal and industrial sewage and waste collection, treatment and disposal facilities within a reasonable time after January 1, 1963.

I hereby find that compliance with the schedule contained in the Recommendations of such Hearing Board is necessary to constitute action reasonably calculated to secure abatement of such pollution within a reasonable time after January 1, 1963.

Dated: June 30, 1960

/s/ Arthur S. Flemming
Secretary

In the Matter of the
POLLUTION OF THE INTERSTATE WATERS
of the
MISSOURI RIVER AND CONNECTING OR TRIBUTARY WATERS IN OR ADJACENT TO THE KANSAS CITIES METROPOLITAN AREA

FINDINGS, CONCLUSIONS, AND RECOMMENDATIONS OF THE HEARING BOARD

The matter of the pollution of the interstate waters of the Missouri River and other interstate waters connecting therewith or tributary thereto in or adjacent to the Metropolitan Area of Kansas City, Kansas, and Kansas City, Missouri, came duly on for hearing at Kansas City, Missouri, on the 13th day

of June, 1960 at 9:00 o'clock a.m., before the board hereinafter named, appointed therefor by the Secretary of Health, Education, and Welfare, pursuant to the call of the Secretary, dated May 17, 1960, under the provisions of the Federal Water Pollution Control Act and the regulations prescribed thereunder, and upon due notice of said hearing given, issued, and served as provided therein. Said hearing was continued on June 14, 15, 16 and 17, 1960, and was then concluded and adjourned. Said Board consisted of the following named persons:

Chester S. Wilson, Chairman
William C. Salome (representing the State of Kansas)
Freeman R. Johnson (representing the State of Missouri)
Sylvester E. Ridge (representing the United States Department of Commerce)
Blucher A. Poole
John S. Samson
Robert C. Ayers, Executive Secretary

The following appearances were entered:

The Surgeon General of the Public Health Service, United States Department of Health, Education, and Welfare, by Murray Stein, Chief, Interstate Enforcement, Water Supply and Pollution Control; Sidney Edelman, Public Health Division, Office of the General Counsel, Department of Health, Education, and Welfare; and Glen J. Hopkins, Regional Engineer, Public Health Service, Region VI;

Missouri Water Pollution Board, by Jack K. Smith, Executive Secretary;

Kansas State Board of Health, by Dwight F. Metzler, Director, Division of Sanitation;

City of Kansas City, Missouri, by Richard H. Koenigsdorf, City Counselor and Benjamin F. Powers, Assistant Counselor;

City of Kansas City, Kansas, by Paul F. Mitchum, Mayor, C. W. Brenneisen, Jr., City Attorney, Carl Hodgkinson, Consulting Engineer, and Truman Schlup, Consulting Engineer;

Fairfax Drainage District of Kansas by Clifford Sharp, Consulting Engineer, and B. Steenhaf;

Kaw Valley Drainage District of Kansas, by R. D. White, and Truman Schlup, Consulting Engineer;

Kansas City Stockyards Company, by Jay B. Dillingham, President;

Bio-Laboratories, Inc., by J. L. Kennedy and Dale Kennedy and by Ralph McKee;

National Cylinder Gas Company, by L. J. Roper, District Manager, R. H. Black, and Ludwig Zengeler, Vice-President;

National Laboratories Corporation, by M. F. Wallace, President;

Phillips Petroleum Company by Reed R. Parker, and Martin E. McMann, Engineering Department, Bartlesville, Oklahoma;

Procter & Gamble Manufacturing Company by Ralph C. Peterson, Chemical Engineer;

Atchison, Topeka, and Santa Fe Railroad Company by J. B. Reeves, Attorney, and C. H. Sandberg, Chief Engineer;

Sinclair Refining Company, by William N. Smith, Terminal Manager;

Swift & Company, by William B. Elson, Jr., Attorney and F. W. Sollo;

Midwest Cold Storage & Ice Corporation by W. Irving Moss, Jr., and John Murphy, Attorney;

General Motors Corporation, Buick, Oldsmobile, and Pontiac Assembly Division, by John Murphy, Attorney;

Corn Products Refining Company, by John R. Moberly, Attorney and H. B. Franey;

American Asphalt Roofing Company, by J. R. Humble, Plant Engineer and A. M. Eberle;

City of North Kansas City, Missouri, by Clifford Sharp, Consulting Engineer;

Chevrolet Assembly Plant, Chevrolet-Kansas Division of General Motors Corporation, by John Murphy, Attorney;

Fisher Body Division, Kansas City Plant, General Motors Corporation, by John Murphy, Attorney;

Schenley Distillers, Incorporated, by W. E. Heidler;

Bendix Aviation Corporation by R. R. Griner and Robert T. Foster;

Prier Brass Manufacturing Company, by Henry J. Hodes;

The Ruberoid Company, by A. M. Eberle, General Superintendent and J. R. Humble, Plant Engineer;

Chamber of Commerce, Kansas City, Missouri, by Herbert Wiggs;

Chamber of Commerce, Kansas City, Kansas, by Ralph Bates.

Now, based on the evidence presented at said hearing the Board makes the following findings, conclusions, and recommendations:

FINDINGS OF FACT

1. All the actions of the Surgeon General of the Public Health Service and the Secretary of Health, Education, and Welfare with respect to the matters involved in this proceeding, as prescribed by the Federal Water Pollution Control Act, were performed and completed prior to the holding of the hearing.

2. The Missouri River, between a point above and near Kansas City, Kansas, and a point below and near the mouth of the Kansas River, flows in a generally easterly direction, constituting a portion of the boundary between the States of Missouri and Kansas. From said last mentioned point said river continues to flow in a generally easterly direction through the State of Missouri to its junction with the Mississippi River. Turkey Creek, including that portion thereof which is now covered and used as a sewer together with the watercourse above and flowing into said covered portion, arises in the State of Kansas, flows in a generally northeasterly direction into the State of Missouri, and thence in a northwesterly and westerly direction into and through the State of Kansas to the junction of said creek with the Kansas River to which said creek is tributary. The Kansas River flows from above said junction in a generally northerly direction through the State of Kansas to the junction of said river with the Missouri River to which said Kansas River is tributary. All of said waters are interstate waters as defined by the Federal Water Pollution Control Act.

3. The City of Kansas City, Kansas, the Fairfax Drainage District of Kansas, the General Motors Corporation, Buick, Oldsmobile, and Pontiac Assembly Division, the Phillips Petroleum Company and each of them have been and now are discharging sewage and industrial and other wastes into said

Missouri River, which discharges originate in the State of Kansas, have the effects hereinafter described, and cause and contribute to substantial pollution of the interstate waters of said river in and adjacent to the State of Missouri, which pollution endangers the health and welfare of persons in said state. The City of North Kansas City, Missouri has been and now is discharging sewage and industrial and other wastes into said Missouri River which discharges originate in the State of Missouri, have the effects hereinafter described, and cause and contribute to substantial pollution of the interstate waters of said river in and adjacent to the State of Kansas, which pollution endangers the health and welfare of persons in said State.

4. The City of Kansas City, Kansas, the Kaw Valley Drainage District of Kansas, the Kansas City Stock Yards Company, the Bio-Laboratories, Inc., the National Cylinder Gas Company, the National Laboratories Corporation, the Procter and Gamble Manufacturing Company, the Atchison, Topeka, and Santa Fe Railroad Company, the Sinclair Refining Company, Swift and Company, and the Midwest Cold Storage and Ice Corporation and each of them have been and now are discharging sewage and industrial and other wastes into said Kansas River, thereafter reaching the Missouri River through said Kansas River, which discharges originate in the State of Kansas, have the effects hereinafter described, and cause and contribute to substantial pollution of the interstate waters of said Missouri River in and adjacent to the State of Missouri, which pollution endangers the health and welfare of persons in said state.

5. The City of Kansas City, Kansas, has been and now is discharging sewage and industrial and other wastes into said Turkey Creek, thereafter reaching said Missouri River through said creek and through said Kansas River, which discharge originates in the State of Kansas, has the effects hereinafter described, and causes and contributes to substantial pollution of the interstate waters of said Missouri River in and adjacent to the State of Missouri, which pollution endangers the health and welfare of persons in said state.

6. The Kansas City Stock Yards Company has been and now is discharging sewage and industrial and other wastes into said Kansas River, thereafter reaching the Missouri River through said Kansas River; which discharge originates in the State of Missouri, has the effects hereinafter described, and causes and contributes to substantial pollution of the interstate waters of said Kansas River and said Missouri River in and adjacent to the State of Kansas, which pollution endangers the health and welfare of persons in said state.

7. The City of Kansas City, Missouri, has been and now is discharging sewage and industrial and other wastes into said Turkey Creek, thereafter reaching the Kansas River through said Turkey Creek and the Missouri River through said Kansas River which discharge originates in the State of Missouri, has the effects hereinafter described, and causes substantial pollution of the waters of said Kansas River and said Missouri River in and adjacent to the State of Kansas, which pollution endangers the health and welfare of persons in said state.

8. The municipalities of Lexington, Glasgow, Boonville, Jefferson City, St. Charles, and St. Louis, and the St. Louis suburban area, respectively, or water utility companies serving the same, use for their public water supplies the waters of said section of the Missouri River below all the discharges aforesaid and are dependent thereon for such supplies. The pollution of said waters from the sources aforesaid causes deterioration of the quality of said waters for the purposes of said public water supplies, in that with said

pollution there is continuous danger of introduction into said waters of disease-producing organisms, and in that there are continuously introduced into said waters with said pollution organic substances and compounds in solution or suspension which provide sustenance for and foster the growth and survival of disease-producing organisms in said waters and which impair the taste and odor thereof for drinking and other purposes. Said agencies provide for the treatment of water taken from said river for their public water supplies in order to render the same safe against the transmission of disease and to minimize or eliminate the taste and odor effects aforesaid. By reason of said pollution the degree and cost of such treatment is substantially increased and the danger of transmission of disease in case of failure, breakdown, or negligent or faulty operation of treatment facilities is substantially increased as compared with what would be the case if said pollution were abated.

9. Said waters of the Missouri River hereinbefore described are used for navigation, boating, fishing, swimming, water skiing, and waterfowl hunting. Said waters of the Kansas River hereinbefore described are used for navigation, boating, fishing, and water skiing. In connection with such uses it is possible for persons engaged therein to contract water-borne diseases from the disease-producing organisms introduced with said pollution, and the use of said water for such purposes is consequently unsafe. All such uses and other beneficial public and private uses of said waters would be promoted and increased if the aforesaid pollution thereof were eliminated or reduced.

10. With the sewage, industrial, and other wastes aforesaid, there are discharged into the waters hereinbefore described human and animal excreta, grease, oil, feathers, and other floating, suspended or dissolved articles or substances of human, animal, or other origin. Some of said articles or substances are offensive and unsightly. Some of said substances cause offensive odors in and about said waters. Some of said substances accumulate on the shores of said waters, and on boats, structures, water intakes, working equipment, fishing gear, and other equipment placed or used in or on said waters, befouling the same and causing trouble in using the same and keeping the same clean. By reason of the facts aforesaid, the articles and substances discharged into said waters as aforesaid frequently cause nuisances therein. By reason of the pollution of said waters as aforesaid, fish of edible species inhabiting the same are sometimes rendered unpalatable. By reason of the pollution of said waters as aforesaid the development and improvement of various types of property and the operation of various business enterprises adjacent to and related to the use of said waters are hampered and retarded.

11. The effects of the pollution of said Missouri River hereinbefore described extend from the points where the discharges of sewage and industrial and other wastes aforesaid enter said river to the junction of said river with the Mississippi River and beyond.

12. If the sewage and industrial and other wastes discharged into said waters as aforesaid were properly, adequately, and effectively controlled and treated, the aforesaid pollution of said waters would be substantially abated and said waters would be substantially restored and improved for the uses and purposes hereinbefore described.

13. Certain of the municipalities and industries hereinbefore named as discharging sewage and industrial and other wastes into the waters hereinbefore described have in good faith manifested an intention to take steps toward abatement of the aforesaid pollution. Nevertheless effective progress

toward abatement of said pollution is not being made by any of said municipalities or industries named as discharging sewage and industrial and other wastes into said waters.

14. Effective, feasible, and reasonable methods for the control and treatment of said sewage and industrial and other wastes are available to each and all of said municipalities and industries hereinbefore named as discharging the same into the waters hereinbefore described. The installation and use of facilities for such control and treatment by said municipalities and industries and each of them are reasonable and equitable measures to abate the pollution of said waters as hereinbefore described. A reasonable time therefor is as stated in the recommendations hereinafter set forth.

CONCLUSIONS

The City of Kansas City, Missouri, the City of Kansas City, Kansas, the City of North Kansas City, Missouri, the Fairfax Drainage District of Kansas, the Kaw Valley Drainage District of Kansas, and the firms or industries hereinbefore named as discharging sewage and industrial and other wastes into the waters hereinbefore described should be required to cease and desist from discharging such sewage and industrial and other wastes into said waters, either directly or indirectly, without proper, adequate, and effective control and treatment, and should be required to provide such control and treatment of such sewage and industrial and other wastes before discharging the same into said waters, so as to abate the pollution hereinbefore described, in accordance with the following recommendations.

RECOMMENDATIONS

It is recommended as follows:

1. That the City of Kansas City, Missouri, the City of Kansas City, Kansas, the City of North Kansas City, Missouri, the Fairfax Drainage District of Kansas, the Kaw Valley Drainage District of Kansas, and the firms or industries hereinbefore named as now discharging sewage and industrial and other wastes into the waters hereinbefore described construct and place in operation such proper, adequate, and effective municipal and industrial sewage and waste collection, treatment, and disposal facilities as will be necessary to abate the pollution of the interstate waters hereinbefore described in accordance with the schedule hereinafter set forth.

2. On or before November 15, 1960, each such firm or industry desiring to be served by the public sewerage system of the appropriate municipality shall advise such municipality of the volume and character of sewage and industrial and other wastes it proposes to discharge to the public system. Should any such firm or industry not desire to be served by such public system, it shall advise such municipality of its desire to be so served on or before the same date.

3. On or before May 1, 1961, each municipality hereinbefore named shall complete the arrangements for the financing of the remedial facilities hereinbefore described.

4. On or before May 1, 1961, each municipality hereinbefore named shall, in writing, instruct its engineer to proceed with final plans and specifications

for proper, adequate, and effective treatment facilities for the sewage and industrial and other wastes to be disposed of through its public system. If such action is not taken by any such municipality, or if such instruction does not provide for service to any industry referred to in Paragraph 1 hereof, each such industry shall initiate detailed planning for its own remedial facilities not later than May 31, 1961.

5. On or before July 1, 1962, final plans and specifications for all such remedial facilities shall be completed and submitted to the state water pollution control agency having jurisdiction for review and approval.

6. On or before January 1, 1963, all such remedial facilities shall be placed under contract for construction, to be completed and placed in operation within a reasonable time.

7. Within 15 days after each date established in this schedule, each municipality and each firm or industry concerned shall advise the Regional Engineer, Region VI, U. S. Public Health Service, as to progress toward compliance with each step of these schedule applicable to it.

Respectfully submitted

In the name of the

HEARING BOARD IN THE MATTER OF POLLUTION OF THE INTERSTATE WATERS OF THE MISSOURI RIVER AND CONNECTING OR TRIBUTARY WATERS IN OR ADJACENT TO THE KANSAS CITIES METROPOLITAN AREA

By

/s/ Chester S. Wilson
Chester S. Wilson, Chairman

ATTEST:

/s/ Robert C. Ayers
Robert C. Ayers, Executive Secretary
Date: June 17, 1960

DID YOU KNOW THAT

Sanitary Engineer Director O. C. Hopkins has been appointed Deputy Chief of Public Health Service's Division of Water Supply & Pollution Control. In this position, he succeeds Arve H. Dahl who has been assigned to a year's tour of duty and training at the Industrial College of Armed Forces in Washington.

Ross H. Walker of Richmond, Virginia took office on July 1 as Chairman of the Ohio River Valley Water Sanitation Commission. He succeeds Maurice E. Gasnell an Attorney for Lawrenceville, Indiana. Mr. Walker has been a Virginia Commissioner on the interstate agencies since its establishment in 1948. He is also a member of the Virginia State Water Control Board.

Ralph C. Graber has been named Chief of the Public Health Service's Air Pollution Engineering Program. He succeeds Sanitary Engineer Director Frank Tetzlaff who has been loaned by the Public Health Service to the International Cooperation Administration to serve as Sanitary Engineering Advisor to the Ministry of Health in Lima, Peru.

Dwight F. Metzler, Director and Chief Engineer of Kansas State Board of Health's Division of Sanitation was elected as Chairman of the Conference of State Sanitary Engineers at the district meeting in Atlanta, Georgia in May.

William Q. Kehr, Executive Director of the Metropolitan St. Louis Sewer District resigned on July 1 to take a position with Public Health Service.

Major General Frank M. Albrecht, South Atlantic Division Engineer, retired from the Army Corps of Engineers on July 21 after more than 37 years as an Army Officer. Colonel Howard A. Morris, District Engineer at Sacramento, California will succeed General Albrecht in September.

Edward Thornton of Manchester, New Hampshire was elected Chairman of the New England Interstate Water Pollution Control Commission at its annual meeting in June.

F. A. Butrico has been named Executive Secretary of the President's Conference on Water Pollution to be held in Washington, D. C. next December.

More bonds were sold for construction of sewers and sewage treatment plants in Pennsylvania than in any other state last year—a total of $52 million.

STEERING COMMITTEE PLANS NATIONAL POLLUTION CONFERENCE

Thirty-five leaders in business, labor, civic organizations and government advising the Public Health Service on the coming National Conference on Water Pollution held their first meeting in Washington June 22-23 and agreed on preliminary plans for the conference program.

In his welcome to the committee, Surgeon General Leroy E. Burney said that the purpose of the conference is to lay the groundwork for a united citizens' attack on the problems of water pollution which can be carried forward in the 1960's.

Plans agreed upon by the advisory committee this week call for a three-day program of panel discussions and plenary sessions designed to assess the problem of water pollution in the United States today, to identify the problems which must be solved, and to reach agreement on methods and means for solving the problem.

Chairman of the advisory committee meetings was Frank A. Butrico, Public Health Service engineer named by Dr. Burney as executive secretary of the conference. Panel discussion groups were decided upon by the

committee to discuss five aspects of water pollution—the nature of the problem; its impact on health, welfare, and the economy; the growing competition for water; the problem of keeping water clean; and the need for additional research.

The National Conference will be held in Washington, December 12-14. More than 1,000 representatives of industry, engineering, medicine, education, research, conservation, government and the public are being invited.

ASCE representative on the conference steering committee is Edward J. Cleary, Executive Director of the Ohio River Valley Water Sanitation Commission.

ST. LOUIS COMPLETES IMPORTANT TRUNK SEWER

One of the most dangerous and long-standing public health menaces in the metropolitan St. Louis area was completely eliminated recently when the last leg of the $1,700,000 Maline Creek Sanitary Trunk Sewer was put into operation at the Walton Road Treatment Plant. With the last link in the trunk line now complete, the 7-mile-long sewer has bottled up all sanitary sewage—most of it raw—from a 25-square-mile area of North St. Louis County that formerly made the creek a vast, open sewer.

Noxious odors and overall pollution in the creek had become so severe throughout the area several years ago that paint on homes bordering the stream actually turned black; numerous cases of illness due to sewage fumes were reported and outdoor activities during the hot summer months were virtually impossible.

ACTIVITIES OF THE SENATE SELECT COMMITTEE

In the May 1960 issue of the Newsletter, the reports and surveys requested by the Senate Select Committee on National Water Resources were listed up to that date. Since then, additional reports have been published as Committee Prints under the general heading "Water Resources Activities in the United States." These recently released reports are as follows:

- Number 6 - Views and Comments of the States
- Number 8 - Future Water Requirements of Principal Water-Using Industries
- Number 10 - Electric Power in Relation to the Nation's Water Resources
- Number 14 - Future Needs for Reclamation in the Western States
- Number 18 - Fish and Wildlife and Water Resources
- Number 21 - Evapo-Transpiration Reduction
- Number 22 - Weather Modification
- Number 24 - Water Quality Management
- Number 30 - Present and Prospective Means for Improved Re-use of Water
- Number 31 - The Impact of New Techniques on Integrated Multiple-Purpose Water Development

Further information concerning these publications may be obtained from Staff Director Theodore F. Schad, Room 3206, New Senate Office Building, Washington 25, D. C.

PENNSYLVANIA WATER BOARD MAKE FIELD INSPECTIONS

The Pennsylvania Sanitary Water Board made a first-hand inspection of water pollution, and waste treatment plants, during a three-day field trip in eastern Pennsylvania in July.

Dr. C. L. Wilbar, Jr., State Health Secretary and Sanitary Water Board chairman, said that "this trip gave Board members an opportunity to see, personally, some of the areas where there is serious water pollution, as well as some of the treatment plants built to keep harmful wastes from the streams."

Among sites visited were: coal strip mines and silt lagoons, raw sewage discharges, industrial waste treatment plants at steel and chemical companies, automatic river monitoring stations and water treatment plants, and a major sewage treatment plant.

Dr. Wilbar said that the trip was similar to that made by the Board one year ago through seven western Pennsylvania counties. He explained that such field trips are designed to give the Board a better understanding of liquid waste control problems, scores of which the Board must act upon at each of its monthly meetings during the year.

COST OF WASTE TREATMENT PLANT RULED A BUSINESS EXPENSE

The Federal District Court for the Middle District of Pennsylvania has allowed immediate deduction of the cost of certain waste treatment facilities installed by direction of the Commonwealth of Pennsylvania through its Sanitary Water Board. In the case of the Woolrich Woolen Mills, Plaintiff v. The United States of America, Defendant, the Court found that:

> "The amount paid by a woolen goods manufacturer for construction of a filtration plant to eliminate pollution from its mill waste was a deductible business expense, not a capital expenditure, where the manufacturer would have been enjoined by court order from discharging its mill waste into a public stream and would thus have had to cease manufacturing operations if it had not constructed the plant, and where the plant erected did not improve, better, extend, increase, or prolong the useful life of the mill property."

NEIWPCC RESEARCH PROGRAM

Under the research program of the New England Interstate Water Pollution Control Commission, a contract has been awarded the Massachusetts Health Research Institute, Inc., for a study involving the chlorination of raw sewage. The project, which was prompted by problems encountered in stream pollution control with storm-water overflows on combined systems of sewerage, will be conducted at the Lawrence Experiment Station of the Massachusetts Department of Public Health. Experiments will be undertaken to determine if bacteriologically acceptable effluents may be obtained by chlorinating comminuted raw sewage and screened raw sewage mixed with storm-water.

The Commission-sponsored project at the University of New Hampshire on the design, operation and effectiveness of the stabilization pond method of treating sewage is being continued. The stabilization ponds at Derry, New

Hampshire, the first of this type of sewage treatment to be constructed in New England, are being used as the basis for the study.

Also continuing is the project being conducted by the Massachusetts Health Research, Inc., at the Massachusetts Institute of Technology and the Lawrence Experiment Station on the treatment of wastes by "package plants" using modifications of the activated sludge process. The final report on the project is scheduled for January 1961.

At the Hall Laboratory of Chemistry at Wesleyan University in Connecticut are Commission projects on treatment methods for wastes from cotton and wool mills, tanneries, dairies and paper plants, in various stages of completion.

CONSTRUCTION CONTRACT AWARDED FOR FIRST SALINE WATER CONVERSION DEMONSTRATION PLANT

A $1,246,250 Office of Saline Water contract for the construction of a one-million gallon per day sea water conversion demonstration plant at Freeport, Texas, has been awarded to the Chicago Bridge and Iron Co.

Contract specifications call for construction and start-up operations to be completed in 330 days; 270 days for construction and 60 days for operational testing before the completed plant is accepted.

The conversion plant at Freeport is the first of five saline water conversion plants authorized by Public Law 85-883. The process to be utilized in this plant, long-tube vertical multiple effect distillation, incorporates several newly developed technical innovations resulting from OSW sponsored research. The plant is expected to produce fresh water from the sea for less than $1 per thousand gallons.

CURBSTONE CLINIC ON MINE DRAINAGE

The second curbstone clinic on mine drainage control, held in Western Pennsylvania June 7-9, attracted sanitary engineering experts from seven states, the USPHS, the Interstate Commission on the Potomac River, and from ORSANCO (under whose auspices the clinic was held).

L. S. Morgan, Chief, Mine Drainage Section, Division of Sanitary Engineering, Pennsylvania Department of Health, described Sanitary Water Board requirements and told of progress in controlling acid mine drainage in the State. A representative of the Pennsylvania Department of Mines outlined reclamation of mined-over areas.

The group inspected a number of backfilled operations, and discussed means of implementing the Acid Mine-Drainage Control Measure.

$12 MILLION GREAT LAKES STUDY PROPOSED

Illinois representatives in Congress urged inclusion of $12 million in federal appropriations to the U. S. Public Health Service for a Great Lakes and Illinois Waterway pollution and water diversion study. Noting the Surgeon General's authority for pollution studies under the federal water pollution control act, the recommendations proposed a six-year study, which would include effects of a 1-year increase in diversion at Chicago, consider also problems of water pollution throughout the Great Lakes.

NEW YORK RATIFIES THE GREAT LAKES BASIN COMPACT

Governor Nelson A. Rockefeller of New York in April signed legislation ratifying the Great Lakes Basin Compact and making the state a member of the Great Lakes Commission. New York's action increases the number of states participating in the Commission to seven, Ohio now being the only state not a member. In Ohio, the governor has appointed a committee of state administrators to study that state's possible entry into the Commission.

NEW CORN-ROT CAUSED BY COLIFORM BACTERIA

A new soft-rot has been discovered on a farm near the Wisconsin River in Wisconsin and in other localities in Wisconsin, Maryland and the Carolinas.

The soft-rot kills corn by attacking the stems so that they eventually break and the plants fall to the ground. Losses due to the disease on the farm where it was discovered amounted to about 10 per cent of the crop.

Research scientists at the University of Wisconsin have discovered that the cause of the disease is a bacterium of the coliform group. The bacterium is one commonly associated with decaying vegetative matter in streams. Serological tests indicate it to be a species of KLEBSIELLA. The bacterium is introduced to the corn by means of overhead spray irrigation.

It was observed that the soft-rot occurs only when river or stream water is sprayed on the plants from above and not when the same water is used in furrow irrigation. A corn breeder in Italy reported that 30 per cent of the plants in a field in his country last year were killed following overhead irrigation with river water.

WATER RESOURCES CONGRESS TO CHART OHIO'S COURSE

Water problems revealed by hearings throughout Ohio will be reviewed at a Water Resources Congress, to be called by Governor Michael V. DiSalle for December 8 and 9 at State Fairgrounds Youth Center, Columbus. Except for a national leader as speaker, the conference will be devoted to the tasks faced by this State in conservation and management of its water resources. The Congress will give everyone an opportunity to become familiar with the problems and participate in plans for their solution. Emphasis will be placed on all of the State's water needs: public and industrial supply, recreation, irrigation, flood protection, flood plain regulation.

The Ohio Water Commission will review its program during the first year of its existence. Reports of the advisory committees will be presented, and possible recommendations to the Governor and the General Assembly on policy and legislation will be discussed. Commission Chairman John A. Slipher, Vice Chairman Herbert B. Eagon, and Executive Secretary Sherman L. Frost have been named as the Congress program committee.

Hearings conducted by the Commission give a sense of urgency to the Water Resources Congress. Leaders in local government and industry in various watersheds have testified in Findlay, Dayton, Youngstown, and Akron. Similar meetings were held in Zanesville June 21 and Columbus June 22. Throughout testimony heard is a dominant note, that present law requiring action within political rather than watershed boundaries is hampering water resource development. An advisory committee on water legislation is working

on this problem. A state-wide program to regulate construction in flood plains also emerges as imperative. Compelling attention is testimony that:

. . . Akron's population and industry by 1980 will make the Cuyahoga River inadequate as a source of water, with the area probably having to depend on Lake Erie.

. . . Youngstown's steel industry will die of thirst and $250 million in planned new industrial construction will not be built unless new sources of water are provided soon.

. . . Summit County's most serious present problem is the Tuscarawas River, needing dredging to prevent floods and permit construction of $2 million in sewers held up by lack of outlet.

STATE WILL MONITOR DESTRUCTIVE DETROIT RIVER POLLUTION

Michigan's Water Resources Commission recently stamped its approval on a $40,000-a-year research and monitoring program on the Detroit River. The action was spurred by recent pollution in the lower Detroit River area where an estimated 12,000 ducks died between mid-March and early April.

Although the $48,000 needed to launch the program full scale for the first year is not available, certain projects will be put into gear as soon as possible. The program will be given high priority in the Commission's next budget request.

The research phase will be sponsored jointly by the Conservation Department and the Water Resources Commission. Wild ducks will be subjected to samples of sewage and industrial waste discharges and chemical pesticides from the river to determine the effects of these substances. These tests will aid in the examination of dead and dying ducks to pinpoint the source causing mortality.

A monitoring project, featuring better communications and coverage, will complement this research. A full-time observer will be assigned to a boat and chartered helicopter to make routine checks of discharges from municipal and industrial sewer outlets and conditions on the Detroit and Rouge Rivers throughout the year. Additional inspections will be made immediately after storms which produce runoff.

PENNSYLVANIA POLLUTION STUDY PLANNED

A nationally-known engineering firm to study the problem of pollution of Spring Creek (Centre County) Pennsylvania and to recommend solutions to the problem has been tentatively selected.

Pollution of Spring Creek, principally from the Pennsylvania State University sewage treatment plant, has affected State Fish Commission hatcheries and famed "Fishermen's Paradise." Local and state agencies are cooperating to find a satisfactory and economical solution, and the matter has led to the formation of an inter-agency working committee, and to a special investigation by the Fisheries Committee of the Pennsylvania House of Representatives.

Among possible solutions to the problem that the engineering firm will be expected to study are:

diverting effluent of the sewage treatment plant to below the hatcheries; pumping the effluent to another watershed; moving the treatment plant downstream, or to another watershed; using oxidation ponds; or providing a more suitable water supply for the hatcheries.

AIR POLLUTION

President Approves Motor Exhaust Study

An Act, originated in the House of Representatives as bill H.R. 8238 "to authorize and direct the Surgeon General of the Public Health Service to make a study and report to Congress . . . of the discharge of substances into the atmosphere from the exhausts of motor vehicles," was passed by both the House and Senate and signed by the President on June 8, 1960, and will henceforth be known as Public Law 86-493. Section 2 of the new law provides that the Surgeon General shall submit his report together with his recommendations, based on the results of the study, "as soon as practicable, but not later than two years" following the passage of the Act. Section 3 defines the term "motor vehicles" as being those propelled by mechanical power and used for transporting passengers or property on a highway."

Illinois Tech Studies Ozone in Stratosphere

An instrument designed to analyze the ozone layer in the stratosphere may unlock many of nature's secrets about weather. Under development for the Air Force Research Division, Geophysics Research Directorate, by Armour Research Foundation of Illinois Institute of Technology, the new "ozone analyzer" will be used to determine the extent of ozone in the atmosphere and variations with seasons.

The instrument, mounted for test purposes on a KC-135 jet tanker, analyzes the ozone by measuring the heat given off by the decomposition process. Based on a conception by Francois Olmer, an ARF scientist, the present instrument uses tiny thermistors—temperature-sensing elements of metallic oxides related to semi-conductors—to measure the heat.

For years it has been suspected that ozone, which is present in the atmosphere between 40,000 and 150,000 feet, plays a prominent role in determining the circulation of the atmosphere. In the stratosphere, absorption of the sun's ultra-violet radiation, for the most part, occurs in the ozone layer which protects us from the radiation's lethal effects.

The sun's absorption and the accompanying processes of ozone formation and decomposition heat a layer in the stratosphere causing turbulence that may extend to the next higher layer—the mesosphere. It is this turbulence and the resulting air currents that are thought to affect the atmospheric circulation. The new "ozone analyzer" is expected to provide information that will cast more light on this phenomenon.

With pilots and potential spacemen flying through or in close proximity to this ozone layer, another need for the ozone analyzer becomes quite apparent. Ozone has a very definite effect on humans as a recent study conducted by Armour Research scientists revealed. It is therefore necessary to monitor the air constantly for ozone content.

Results of the ARF ozone toxicity study indicate that in sensitive persons, concentrations as low as two parts ozone per million of air may cause severe

lung irritation in less than an hour. The ozone affects the soft tissue of the respiratory tract and, the longer the exposure or the higher the concentration, the deeper the tissue damage.

U. S. Supreme Court Hands Down Two Great Lakes Decisions

The U. S. Supreme Court in April upheld application of the Detroit anti-smog ordnance against shipping in the Detroit River. The case had been appealed to the high tribunal by the Huron Portland Cement Company after the State Courts rejected the company's request for an injunction against enforcement of the ordnance claiming that the boilers of its vessels are operated under federal regulations (coast guard license).

Another ruling by the Supreme Court in May held that the U. S. may prohibit the discharge of liquid industrial wastes into navigable waters. The case involved waste materials in liquid form from steel mill operations into the Calumet River. The court said the deposit of the waste, even though small, reduced the navigable capacity of the river and is not exempt under the Federal Rivers and Harbors Act of 1899.

New Publications Available on Air Pollution

"Cleaner Air for North Carolina" by Billy C. Blakeney and Marvin D. High reports on a recent survey and appraisal of air pollution problems in that State undertaken jointly by the North Carolina Board of Health and the Public Health Service. This is a 64-page booklet with photographs and an attractive colored cover. Also available is a reprint of an article from the January 1960 issue of Fortune Magazine, "The Noisome Problem of Car Fumes" by George A. W. Boehm. "Air Monitoring and Sampling Networks"—Proceedings of the seminar held at Robert A. Taft Sanitary Engineering Center, November 23-24, 1959—is a 136-page (8-1/2" x 11") bound volume. Information concerning these publications may be obtained from Chief, Air Pollution Engineering Program, Public Health Service, Washington 25, D. C.

Nation's Capitol Has New Plan to Curb Auto Fumes

Engineer Commissioner A. C. Welling recently announced plans to help reduce air pollution from traffic exhaust fumes in Washington. General Welling's proposals include three major provisions. The first would require that all new motor vehicles registered in the District beginning with the 1962 models be equipped with a "blow-by" device to cut down escaping combustion gases. The second would set a three-minute motor idling limit (except in below-freezing weather) for vehicles not in the stream of traffic. The final provision would forbid operation of a vehicle where there is "visible" smoke or fumes from the exhaust for more than one-tenth of a mile. These proposals will be considered at public hearings late in August and, if approved, will make Washington the first city to require the "blow-by" device.

NUCLEAR ENERGY

RAD. Health Specialist Shortage Discussed at Princeton

A three-day meeting to explore methods of reducing the nationwide shortage of radiological health specialists was held in Princeton, New Jersey on

August 2. The meeting attracted over 100 representatives of universities, professional societies and government health agencies.

In calling the meeting, Dr. Leroy E. Burney, Surgeon General of the Public Health Service, said the objective of the symposium was to explore the various training needs of radiological health programs in Federal, State and local health agencies. He said these programs need many hundreds of trained specialists in connection with the rapid expansion of the use of atomic energy, the increasing use of X-ray and other radiation sources in the healing arts and in industry.

Dr. Burney pointed out that the Public Health Service has conducted short-term technical training programs in radiological health for the past ten years. National leaders, however, believe that more attention must be given to increasing university efforts, at both the graduate and undergraduate levels, in the teaching of radiological science courses.

The symposium featured organized discussions about the types of curricula required for professional preparation for research and health protection in the field of radiation.

WATER SUPPLY AND POLLUTION CONTROL

Analysis of SED Publications

In the March issue of the News an analysis was given of material published in the Journal during the past four years. This analysis presented tabular information as to whether an author was a non-member of the ASCE and, if a member, what grade he held at the time of publication.

The following table summarizes the papers published by type of occupational affiliation for the same period (1956-1959) as the March summary.

Summary of Papers Published
Type of Author Affiliation*
Sanitary Engineering Division Journal
1956 - 1959

Article Classification	Government			Education	Consulting	Industry & Utilities	Misc.
	Federal	State	Other				
Air Pollution	5	2	3	4	2	1	-
Education	6	-	1	8	2	1	-
Industrial Wastes	11	-	1	6	2	1	-
Public Health	2	-	1	1	1	-	-
Refuse	1	-	-	1	3	-	-
Sewage	11	-	10	12	11	4	1
Stream Pollution	11	2	3	5	2	2	1
Water	12	2	4	5	8	1	1
Miscellaneous	3	-	1	10	2	2	1
Total	62	6	24	52	33	12	4
Per Cent of Total	32	3	13	27	17	6	2

* Does not include Committee Reports

NEW DIRECTORY IS AVAILABLE TO MEMBERS

The 1960 Directory is now available to members on request. The Directory lists the entire membership of the Society, giving the membership grade, position, and mailing address of each. In addition, there is a complete listing of the Honorary Members, past and present, and the Life Members. A useful geographical listing of the members is also included.

It goes without saying that the information contained in the Directory is of value to every member, and every member can obtain this valuable information. To receive your free copy of the Directory simply fill out the coupon below. Prompt delivery depends on prompt return of the coupon.

The Society publishes the membership Directory every other year. The next edition will be issued in 1962.

DIRECTORY 1960

ASCE members are entitled to receive, free of charge, the 1960 ASCE Directory. To obtain the directory simply clip this coupon and mail to: American Society of Civil Engineers, 33 West 39th Street, New York 18, N. Y.

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ASCE-WPCF JOINT SEWER MANUAL ISSUED

A major addition to the ASCE series of Manuals of Engineering Practice is now available in a new volume entitled "Design and Construction of Sanitary and Storm Sewers." Identified as No. 37, this publication is the result of several years of joint effort by the Sanitary Engineering Division of ASCE and the Water Pollution Control Federation (formerly the Federation of Sewage and Industrial Wastes Associations).

The twelve-chapter sewer manual contains 283 pages, over 100 illustrations, 24 tables, and more than 100 references. As the first extended collection of information on the subject, it will make a valuable reference in an important phase of wastewater technology. Individual subjects covered include organization and administration of sewer projects, surveys and investigations, quantity of sanitary sewage and storm water, hydraulics of sewers, design of sewer systems, appurtenances and special structures, materials for sewer construction, structural requirements, construction plans and specifications, construction methods, and pumping stations.

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